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TEST FILLS FOR ROCK-FILL DAMS.(U)
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AD-A035 981

TEST FILLS FOR ROCK-FILL DAMS

ARMY ENGINEER WATERWAYS EXPERIMENT STATION
VICKSBURG, MISSISSIPPI

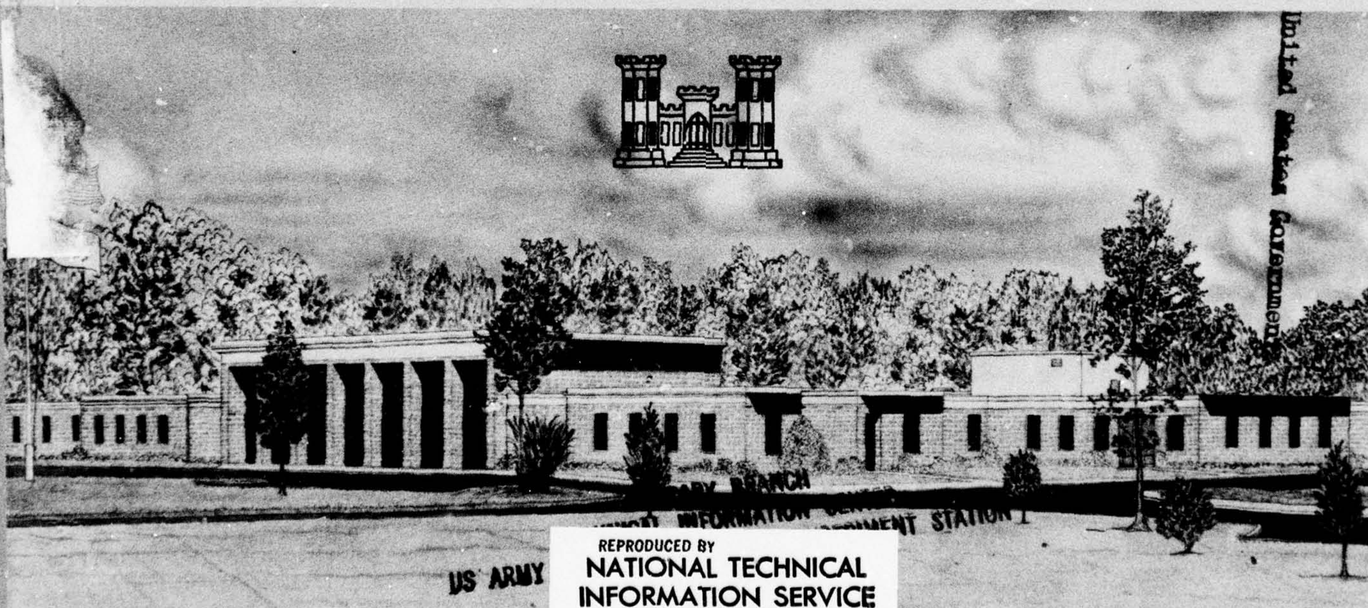
MARCH 1973

MISCELLANEOUS PAPER S-73-7

TEST FILLS FOR ROCK-FILL DAMS

by

D. P. Hammer, V. H. Torrey III



March 1973

Sponsored by Office, Chief of Engineers, U. S. Army

Conducted by U. S. Army Engineer Waterways Experiment Station
Soils and Pavements Laboratory
Vicksburg, Mississippi

Unclassified
Security Classification

DOCUMENT CONTROL DATA - R & D

(Security classification of title, body of abstract and indexing annotation must be entered when the overall report is classified)

1. ORIGINATING ACTIVITY (Corporate author) U. S. Army Engineer Waterways Experiment Station Vicksburg, Miss.	2a. REPORT SECURITY CLASSIFICATION Unclassified	2b. GROUP

3. REPORT TITLE

TEST FILLS FOR ROCK-FILL DAMS

4. DESCRIPTIVE NOTES (Type of report and inclusive dates)
Final report

5. AUTHOR(S) (First name, middle initial, last name)
David P. Hammer
Victor H. Torrey III

6. REPORT DATE	7a. TOTAL NO. OF PAGES	7b. NO. OF REFS
March 1973	65	0
8a. CONTRACT OR GRANT NO.	9a. ORIGINATOR'S REPORT NUMBER(S)	
b. PROJECT NO.	Miscellaneous Paper S-73-7	
c.	9b. OTHER REPORT NO(S) (Any other numbers that may be assigned this report)	
d.		

10. DISTRIBUTION STATEMENT

Approved for public release; distribution unlimited.

11. SUPPLEMENTARY NOTES	12. SPONSORING MILITARY ACTIVITY
	Office, Chief of Engineers, U. S. Army Washington, D. C.

13. ABSTRACT Data from 14 Corps of Engineers' (CE) rock test fill projects are summarized, and six of these projects are analyzed in detail. Variables most often investigated in these test fills were (a) lift thickness, (b) roller type, (c) number of roller passes, and (d) rock gradation. Measured parameters were (a) compaction, (b) permeability, and (c) grain-size distribution, both before and after compaction. The vibratory roller generally gave the best compaction, but also caused a substantial amount of surficial breakage for most rock types. It was found in most cases that better results were obtained with a vibratory roller when compacting material with the fines (sizes less than ± 3 in.) removed. The possibility of using the Los Angeles abrasion test to predict rock breakage was explored. However, no conclusions could be drawn due to the lack of data and to the diversity of testing methods used in the projects studied. Recommended procedures for (a) planning and design, (b) construction, (c) measurements and observations, and (d) evaluation of results of future test fills are given.

DD FORM 1 NOV 65 1473 REPLACES DD FORM 1473, 1 JAN 64, WHICH IS OBSOLETE FOR ARMY USE.

Unclassified
Security Classification

Unclassified

Security Classification

14. KEY WORDS	LINK A		LINK B		LINK C	
	ROLE	WT	ROLE	WT	ROLE	WT
Rock fills						
Rock-fill dams						
Test fills						

Unclassified

Security Classification

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FOREWORD

The study reported herein was authorized by the Office, Chief of Engineers, in a letter (ENGCW) dated 31 January 1969, subject: Special Studies Group for Civil Works Soils Problems, ES 537.

This study was conducted during the period March 1969 - July 1972 by the U. S. Army Engineer Waterways Experiment Station (WES) under the general direction of Messrs. J. P. Sale, Chief; Mr. S. J. Johnson, Special Assistant; and Mr. J. R. Compton, Chief, Soil Mechanics Division, Soils and Pavements Laboratory. Principal engineers conducting the study and analyzing results were Messrs. W. E. Strohm, Jr., former Chief, Engineering Studies Section; D. P. Hammer, and V. H. Torrey III, Soil Mechanics Division, Soils and Pavements Laboratory. This report was prepared by Messrs. Hammer and Torrey.

Directors of WES during the preparation and publication of this report were COL Levi A. Brown, CE, and COL Ernest D. Peixotto, CE. Technical Director was Mr. F. R. Brown.

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CONVERSION FACTORS, BRITISH TO METRIC UNITS OF MEASUREMENT

British units of measurement used in this report can be converted to metric units as follows:

Multiply	By	To Obtain
inches	2.54	centimeters
feet	0.3048	meters
miles	1.609344	kilometers
square inches	6.4516	square centimeters
cubic yards	0.764555	cubic meters
tons (2000 lb)	907.185	kilograms
pounds per square inch	0.070307	kilograms per square centimeter
pounds per cubic foot	16.0185	kilograms per cubic meter

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SUMMARY

Data from 14 Corps of Engineers' (CE) rock test fill projects are summarized, and six of these projects are analyzed in detail. Variables most often investigated in these test fills were (a) lift thickness, (b) roller type, (c) number of roller passes, and (d) rock gradation. Measured parameters were (a) compaction, (b) permeability, and (c) grain-size distribution, both before and after compaction.

The vibratory roller generally gave the best compaction, but also caused a substantial amount of surficial breakage for most rock types. It was found in most cases that better results were obtained with a vibratory roller when compacting material with the fines (sizes less than $\pm$ 3 in.) removed.

The possibility of using the Los Angeles abrasion test to predict rock breakage was explored. However, no conclusions could be drawn due to the lack of data and to the diversity of testing methods used in the projects studied.

Recommended procedures for (a) planning and design, (b) construction, (c) measurements and observations, and (d) evaluation of results of future test fills are given.

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TEST FILLS FOR ROCK-FILL DAMS

PART I: INTRODUCTION

Background

1. Embankments composed largely of rock fill are becoming more common, primarily because of increased heights of dams, poor foundation conditions that rule out concrete dams, and vast improvements in modern construction equipment that now make excavation of rock and placement of rock fill a much more economical procedure than in the past. There exist no well-established criteria for construction and control testing methods best suited to a rock-fill embankment, and the technology for predicting the behavior of rock as a construction material is extremely limited. Therefore, the construction of a test fill can be of great value and in some cases is indispensable. The information obtained from such a test fill helps the designer to better define the effect of variables, such as rock gradation, lift thickness, compaction equipment, and number of passes, on the quality of the compacted rock fill. An improved knowledge of these variables allows the designer to select a compaction procedure that will produce a rock fill of desired quality at a minimum cost. It should also be noted that even though this study is not specifically concerned with test quarries, they are very useful and should be employed to complement test fills if at all possible. The proper utilization of a test quarry will enable a designer to better define the type or types of material that will be used for the construction of the actual embankment. A test fill represents a sizeable expenditure in itself; thus it is imperative that the maximum amount of information be derived. This study was designed to better achieve this end by

summarizing the Corps of Engineers' (CE) experience in the construction and testing of rock test fills. The work was accomplished under Engineering Study (ES) 537, Special Studies Group for Civil Works Soil Problems.

Purpose and Scope

2. The primary objectives of this study were to summarize selected test fill investigations for CE rock-fill dams, to analyze current procedures used, and to make recommendations for future test fills. To accomplish these objectives, six CE rock test fills (Beltzville, Laurel, Gathright, Cougar, New Melones, and Gillham) are described and test data are evaluated. Based on an analysis of these six test fills, recommendations are suggested for future test fills. A summary of pertinent information on these six test fills and eight other CE test fills is also given as a ready reference of information available for particular rock types. In addition, a brief summary of two rock test fill programs conducted by other agencies is given in Appendix A. Appendix B describes attempts to correlate results of Los Angeles abrasion tests with the amount of degradation occurring during actual handling and compaction. Appendix C contains a detailed procedure for performing in situ density tests in compacted rock fill by the water-displacement method.

3. This report is limited to those fills composed of rock that would not break down into a tight impervious mass (resembling a soil) during compaction but, to a certain degree, would retain its original identity even though substantial breakage may occur.

PART II: SUMMARY OF DATA ON SELECTED CE ROCK TEST FILLS

Beltzville Dam, Philadelphia District

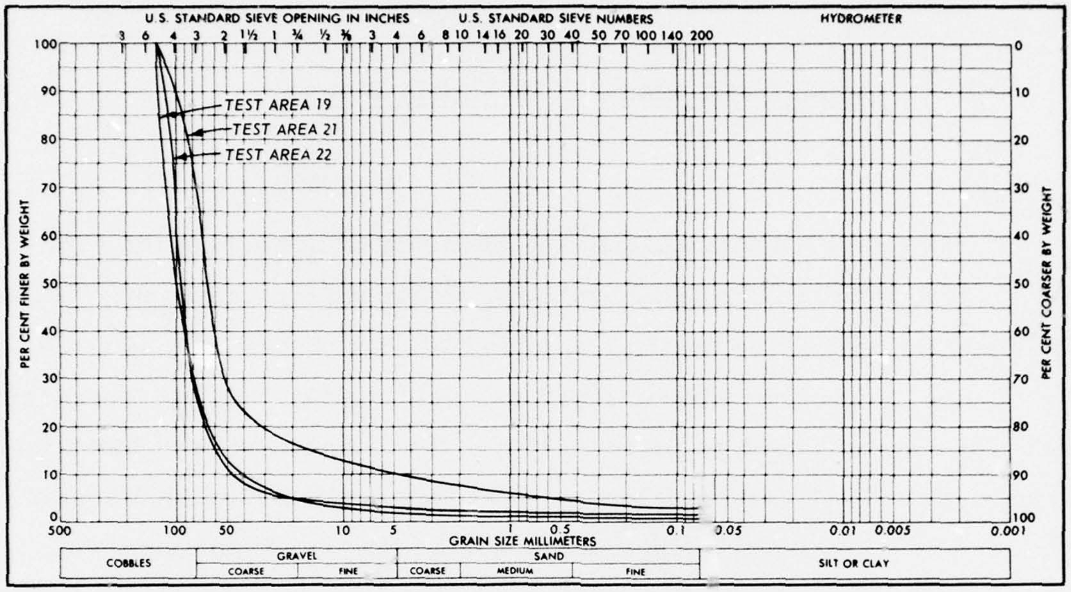
General

4. This test fill was planned and constructed in conjunction with the design of Beltzville Dam and Reservoir, Pohopoco Creek, Pennsylvania. Construction took place during the period 25 July to 30 September 1966. The design, supervision of construction, field testing, and all measurements were performed by personnel from the Foundations and Materials Branch of the Philadelphia District. The information contained herein was largely taken from Design Memorandum No. 11, Supplement No. 1, "Embankment and Foundations, Beltzville Dam and Reservoir," February 1967, prepared by the Philadelphia District.

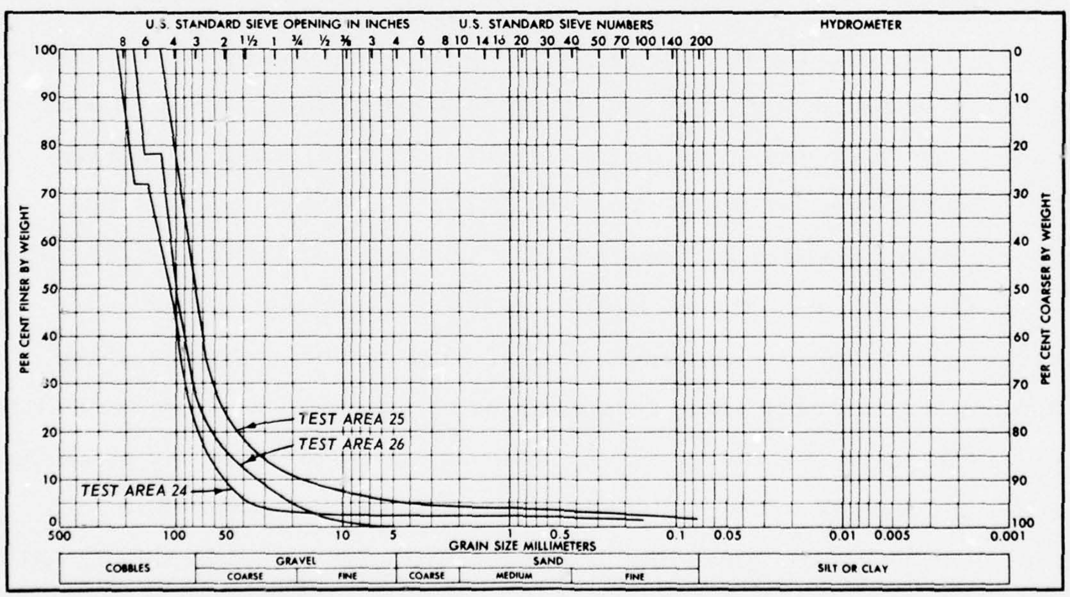
Rock type

5. The rock-fill materials placed in the test fill consisted of shales of the Mahantango formation and included a red-brown, highly weathered shale; a brown-green-gray, partly weathered shale; and a dark gray, hard, dense, relatively unweathered shale. For the purpose of this report, only the latter two harder types of shale were considered. When loosened by blasting, the partly weathered shale produced elongated angular pieces up to a maximum size of 15 in.* The gradation of the angular, unweathered shale fragments after blasting was very similar to that of the partly weathered shale; however, occasional slabby pieces up to 36 in. in length were produced. Typical gradation curves for both types of rock before spreading on the test fills are shown in fig. 1. All oversize fragments were removed from the fill prior to compaction. Breakage of the rock under handling and compaction varied according to

* A table of factors for converting British units of measurement to metric units is presented on page xi.



a. PARTLY WEATHERED SHALE



b. RELATIVELY UNWEATHERED SHALE

Fig. 1. Typical gradations of material before spreading Beltzville test fill

the type of equipment used. A field evaluation of particle breakage caused by spreading and compaction is given in table 1.

Description of test fill

6. In order to differentiate between the two types of material, two construction zones were used: Zone C consisted entirely of the partly weathered shale, while Zone D was constructed using relatively unweathered shale. Each zone was further divided into four test areas to evaluate different compaction methods. A plan and profile of the test fill area is shown in fig. 2.

Construction

7. The foundation for the test fill consisted of a highly weathered shale which was stripped to the desired founding elevation and compacted by ten passes of a vibratory roller.

8. All fill material was trucked from the quarry, dumped at the leading edge of the lift being placed, and spread to the required thickness with a tracked bulldozer. Level readings were made to ensure the proper loose lift thickness. Compaction was then carried out according to the schedule given in figs. 3 and 4 and as summarized in table 2. Equipment specifications are also given in figs. 3 and 4. Estimated speed of the vibratory roller was 1.5 mph; all other rollers worked at approximately 3 mph.

Tests and measurements

9. Procedure

a. Field density tests. The initial testing schedule for the test fill called for one density test in each test area. This was later modified in Test Zone D, where the test trenches indicated that the best compaction of the relatively unweathered shale was in test area 24 and that testing of other portions of Zone D would not be required. Modifications were also made to the original testing schedule in Zone C by concentrating the tests in those areas where a more compact fill was indicated. The density testing was performed by excavating the

Table 1

Visual Evaluation of Particle Breakage, Beltzville Test Fill

Operation	Breakage Evaluation
Partly Weathered Shale	
Spreading with tracked bulldozer	Extensive surface breakage
Compaction with shale breaker roller	Larger pieces reduced
Compaction with rubber-tired roller (following shale breaker)	Additional breakage on surface
Relatively Unweathered Shale	
Spreading with tracked bulldozer	Limited surface breakage
Compaction with rubber-tired roller	Limited surface breakage
Compaction with vibratory roller	Extensive surface breakage

* Note: Description of equipment is given in figs. 3 and 4.

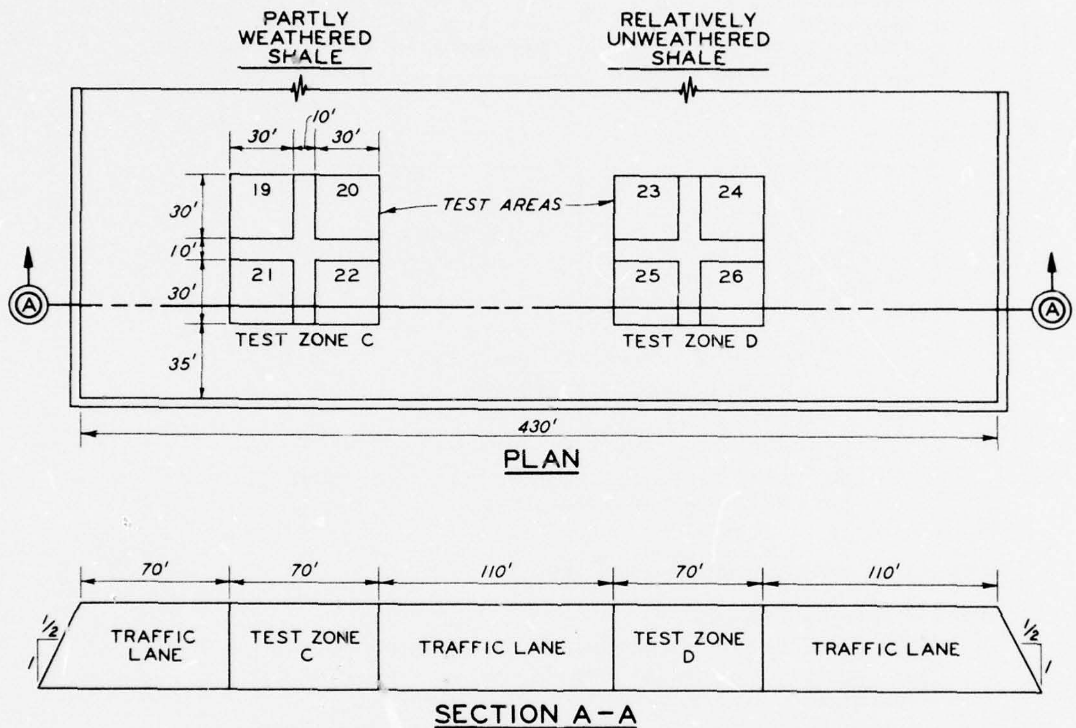
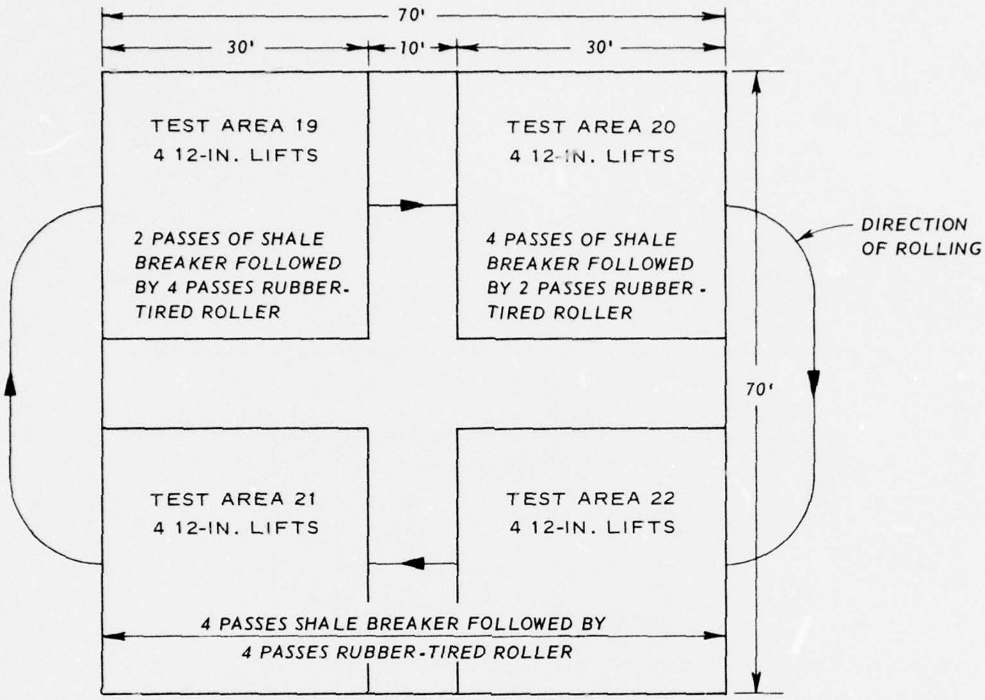


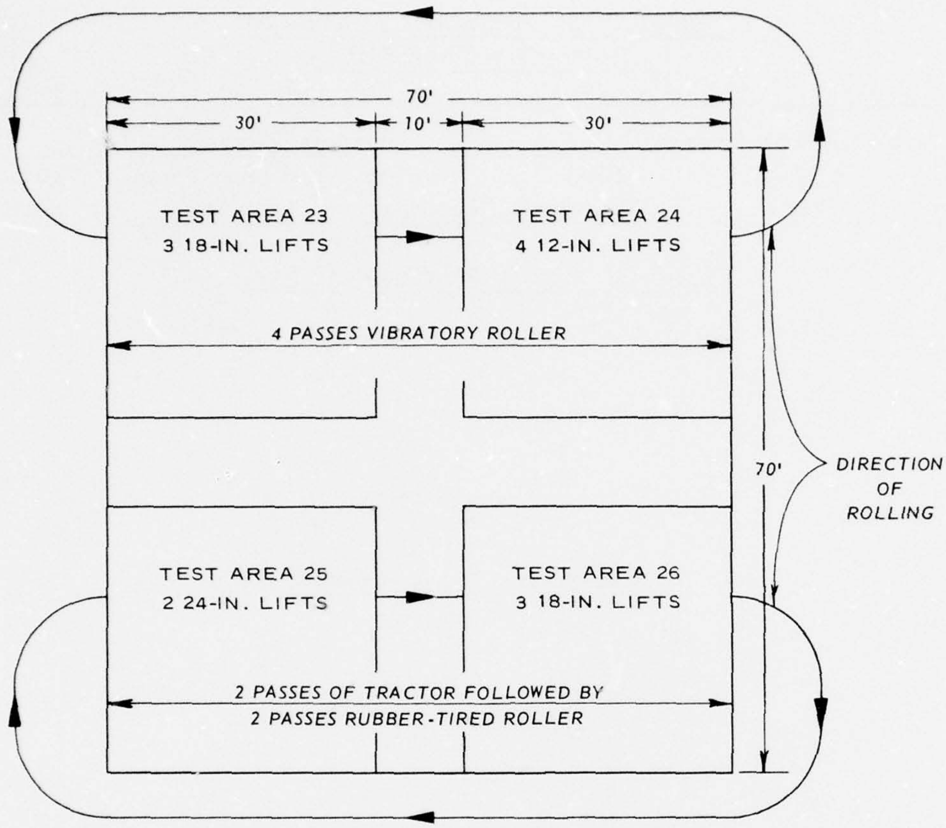
Fig. 2. Plan and profile of Beltzville test fill



NOTE: ALL LIFT THICKNESSES REFER TO THE LOOSE OR UNCOMPACTED THICKNESS.

1. MATERIAL: PARTLY WEATHERED SHALE
2. COMPACTION EQUIPMENT:
   A. SHALE BREAKER, FERGUSION-GEBHARD MODEL 22, FULLY BALLASTED WITH CHISELTIPS HAVING A FACE AREA OF 1-1/2 SQ IN. AND A 3750 PSI TIP PRESSURE
   B. RUBBER-TIRED ROLLER, 50-TON BROS, FULL BALLASTED, FOUR PNEUMATIC TIRES ABREAST AT 90 PSI, 25,000-LB WHEEL LOAD
3. ALL SPREADING OF MATERIAL AND TOWING OF COMPACTION EQUIPMENT WAS WITH AN ALLIS-CHALMERS HD-20 TRACKED BULLDOZER

Fig. 3. Compaction schedule, Zone C, Beltzville test fill



1. MATERIAL: RELATIVELY UNWEATHERED SHALE
2. COMPACTION EQUIPMENT:
   A. VIBRATORY ROLLER - FERGUSON MODEL 230, WORKING WEIGHT 23,500 LB, FREQUENCY OF VIBRATION VARIED FROM 1100 TO 1300 VPM
   B. RUBBER-TIRED ROLLER AND HD-20 BULLDOZER AS DESCRIBED PREVIOUSLY IN FIG. 3
3. THE HD-20 BULLDOZER WAS USED FOR SPREADING AND AS PRIME MOVER FOR COMPACTION EQUIPMENT

Fig. 4. Compaction schedule, Zone D, Beltzville test fill

Table 2

Schedule of Variables Evaluated
Beltzville Test Fill

Test Area	No. - Thickness of Lift, in.	No. of Passes: Shale Breaker	Crawler Tractor	Rubber-Tired Roller	Vibratory Roller
		Zone C, Partly Weathered Shale			
19	4-12	2	-	4	-
20	4-12	4	-	2	-
21	4-12	4	-	4	-
22	4-12	4	-	4	-
		Zone D, Relatively Unweathered Shale			
23	3-18	-	-	-	4
24	4-12	-	-	-	4
25	2-24	-	2	2	-
26	3-18	-	2	2	-

material inside a 36- by 36-in. wooden template and measuring the volume with either sand or water. Where water was used as the displacement medium, a thin plastic membrane was carefully fitted to the sides of the hole prior to filling. Excavated material for these tests averaged approximately 18 cu ft (corresponding to an approximate 2-ft depth). One test in the partly weathered shale utilized a 15- by 15-in. wooden template rather than the larger 36-in. template. Comparative tests were also made using a nuclear density and moisture probe.

b. Mechanical analyses. Gradation tests were performed on representative samples taken before spreading and on samples made up of material excavated for the field density tests, which represented material after compaction. Sample weights for each test varied between 600 and 800 lb.

c. Settlement measurements. The typical layout used for taking level readings over each test area is shown in fig. 5. Strings with markings at 5-ft centers were stretched from side stakes, and the grid points thus located were sprayed with red paint for identification. Steel plates were placed on the prepared foundation in order to provide a reference elevation before and after test fill operations to ensure that final computed settlements would not include any foundation settlement. Readings taken consisted of initial readings prior to spreading, readings to establish proper lift thickness, and readings after every two passes of the compaction equipment.

d. Inspection trenches. A 5-ft-wide inspection trench was excavated over a 25-ft length for the full fill depth in each test area. These trenches enabled visual observations to be made of the tightness of the fill and the bonding between lifts. A thin coating of lime had been placed on the surface of each compacted lift prior to the spreading of the next lift to facilitate layer identification in the walls of the trenches.

10. Results

a. Field density tests. A number of density tests were made upon completion of each test area. The results of these tests are presented in table 3. An examination of these

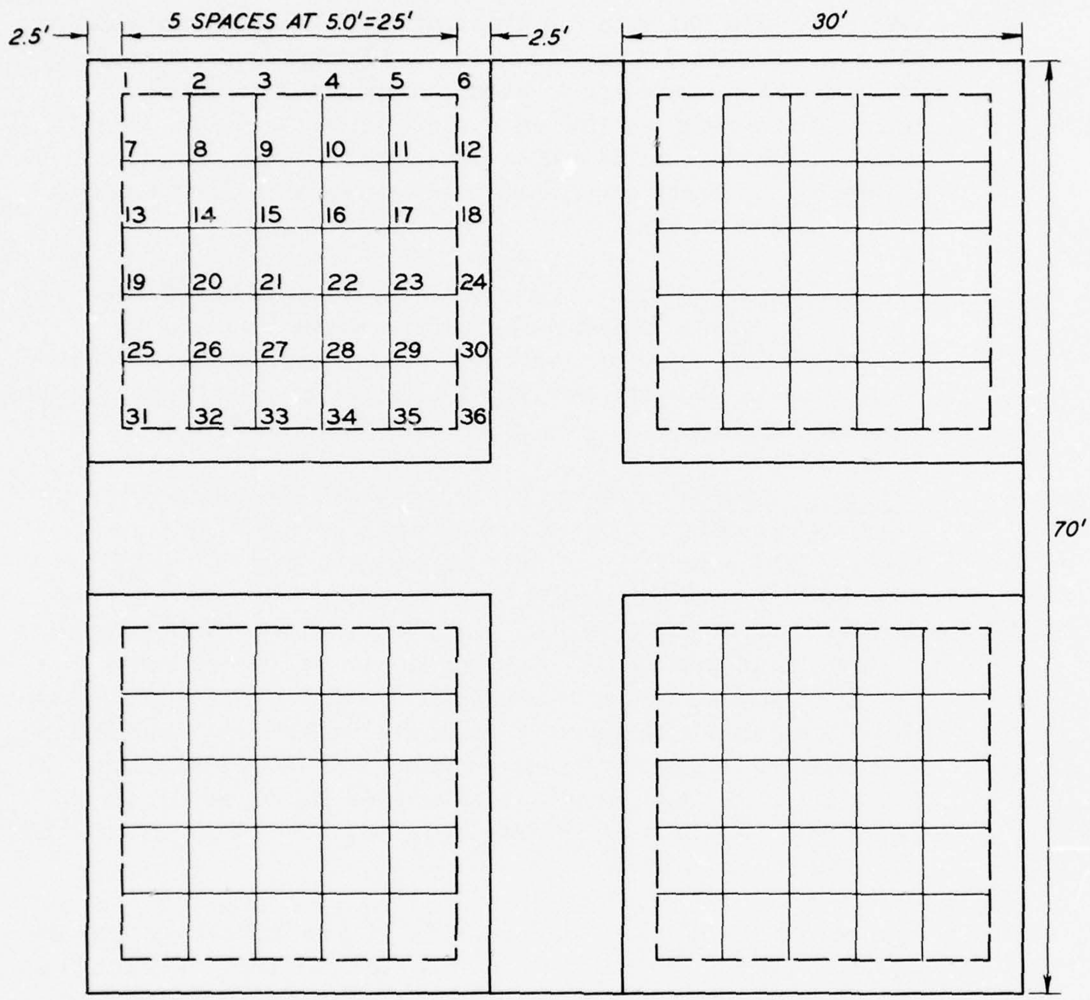


Fig. 5. Typical grid for level readings Beltzville test fill

Table 3

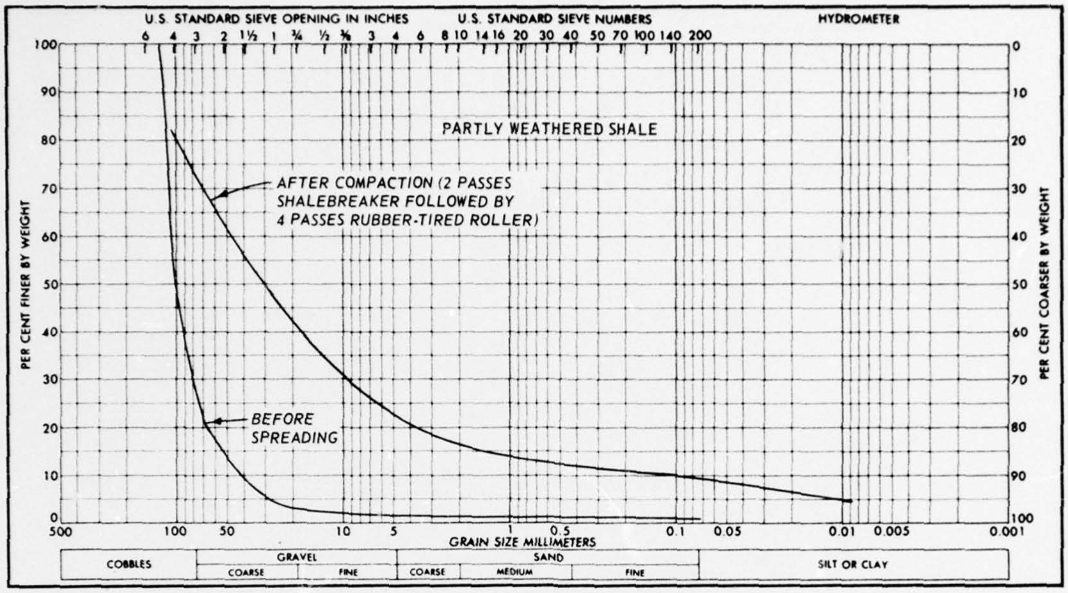
Beltzville Test Fill, Field Density Test Data

Test Area	Compaction	No. of Lifts	Lift Thickness in.	Test Method	Water Content w, %	Dry Density pcf
Zone C, partly weathered shale						
19	2 passes shalebreaker plus 4 passes 50-ton rubber-tired roller	4	12	36-in. template - sand	8.7	119.7
				36-in. template - water	9.0	122.4
				Nuclear	13.8	105.0
				Nuclear	12.0	112.0
20	4 passes shalebreaker plus 2 passes 50-ton rubber-tired roller	4	12	Nuclear	11.6	107.5
21	4 passes shalebreaker plus 4 passes 50-ton rubber-tired roller	4	12	36-in. template - sand	8.9	120.0
				36-in. template - water	8.6	126.6
				15-in. template - sand	8.8	117.8
				Nuclear	10.0	109.5
				Nuclear	13.6	99.0
22	4 passes shalebreaker plus 4 passes 50-ton rubber-tired roller	4	12	36-in. template - sand	10.5	123.8
				Nuclear	11.8	102.0
Zone D, relatively unweathered shale						
23	4 passes vibratory roller	3	18	Nuclear	9.2	125.0
24	4 passes vibratory roller	4	12	36-in. template - sand	3.9	127.1
				Nuclear	8.8	125.0
25	2 passes dozer plus 2 passes 50-ton rubber-tired roller	2	24	Nuclear	11.3	111.5
26	2 passes dozer plus 2 passes 50-ton rubber-tired roller	3	18	Nuclear	8.2	110.5

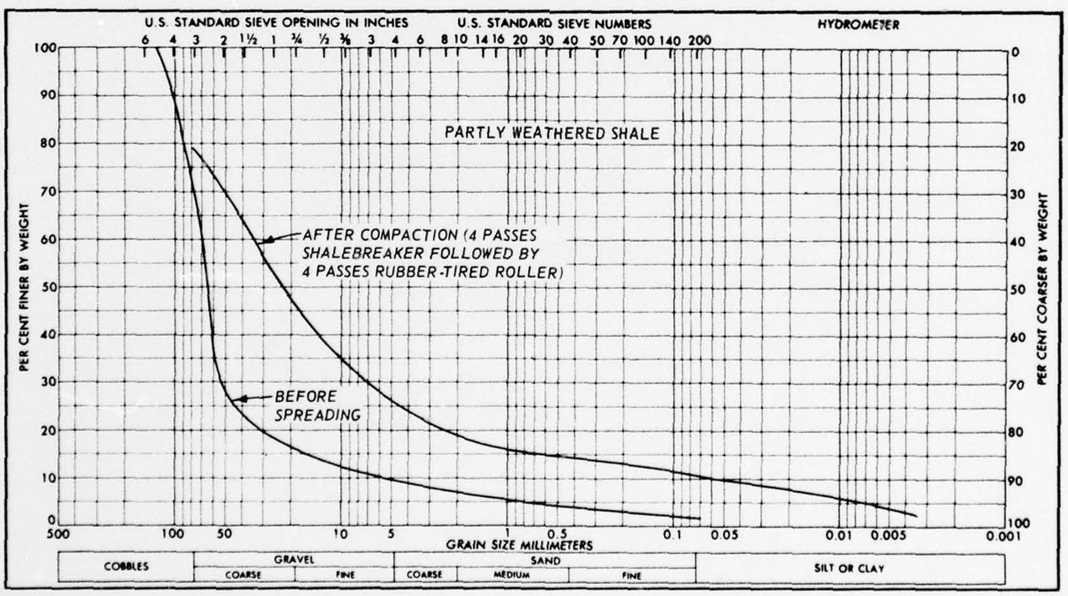
limited data reveals moderately close agreement between the use of sand and water as the displacement medium for the large volume (36- by 36-in.) tests, although the water displacement gave the higher densities. Approximately the same amount of work (28 man-hours) was required to perform either the water or sand density test. The water method was a little easier, however, because it required less physical work; but the accuracy of the test was controlled by the amount of care exercised in fitting the plastic membrane to the sides of the test hole. The nuclear meter gave densities in Test Zone C that were obviously too low and, apparently so, in test areas 25 and 26 in Test Zone D. This discrepancy is also reflected in the water content results. Only in areas 23 and 24, where the vibratory compactor was used, did the nuclear meter give densities high enough to be comparable with the large-volume displacement-type tests, although the data were somewhat scanty for these test areas. This is possibly explained by the fact that the vibratory roller left a much smoother surface than did the other rollers, thus providing a better seat for the nuclear apparatus with fewer air spaces at the contact surface.

b. Mechanical analyses. Results of mechanical analyses are shown in figs. 6 and 7. Each plot contains representative gradations of material before spreading and after compaction in order to demonstrate the degradation that occurred due to spreading and compaction. As is evidenced by these data, considerable degradation took place in both the partly weathered shale and the relatively unweathered shale during spreading and compaction. However, the distribution of fines throughout the lift is not discernible since the gradation curves represent total samples.

c. Settlement measurements. Plots of the percent settlement for each lift versus the number of compaction equipment passes are shown in fig. 8 (Test Zone C) and fig. 9 (Test Zone D). By expressing the settlement obtained in terms of a percentage of the loose lift thickness (nominal loose lift thickness was used in this case as the values of actual loose lift thickness were unavailable), a comparison of the settlement characteristics of lifts of different thicknesses can be made. If actual settlement were plotted instead of percent

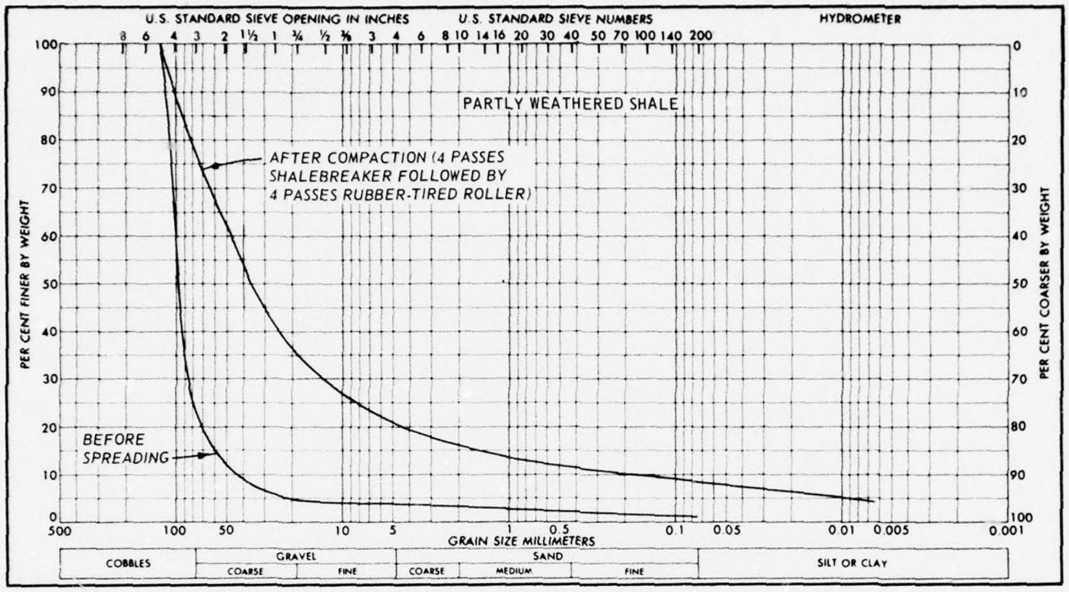


a. TEST ZONE C, AREA 19, LIFT 3

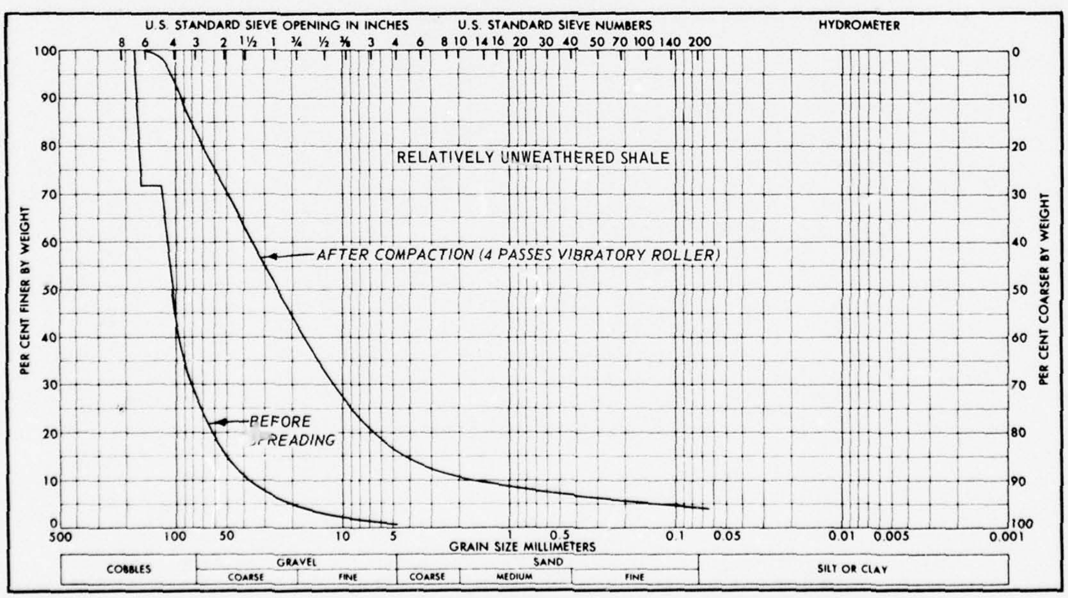


b. TEST ZONE C, AREA 21, LIFT 2

Fig. 6. Before spreading and after compaction gradation curves Test Zone C Beltzville test fill



a. TEST ZONE C, AREA 22, LIFT 1



b. TEST ZONE D, AREA 23, LIFT 3

Fig. 7. Before spreading and after compaction gradation curves Test Zones C and D Beltzville test fill

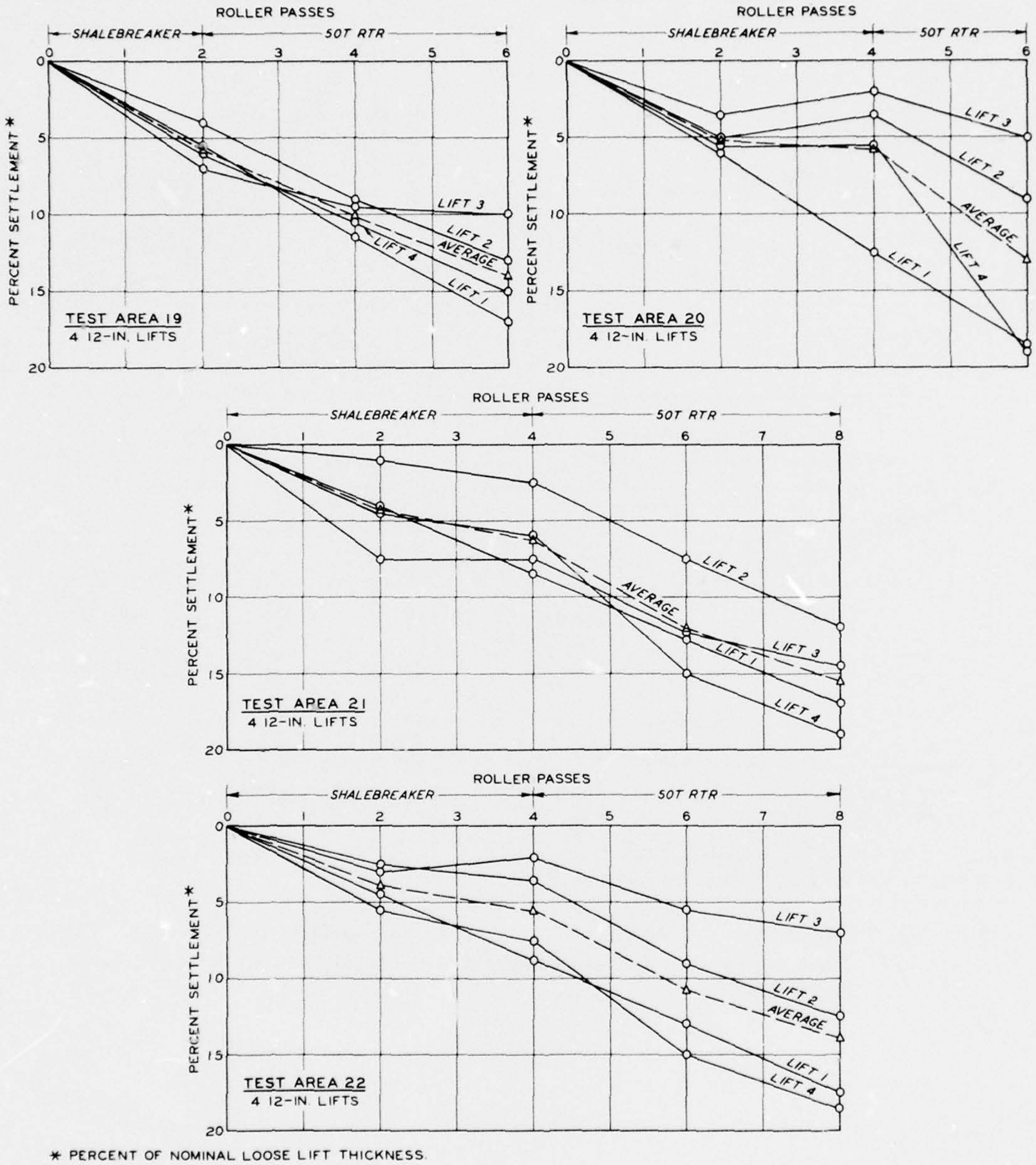


Fig. 8. Percent settlement vs roller passes for each lift Test Zone C partially weathered shale Beltzville test fill

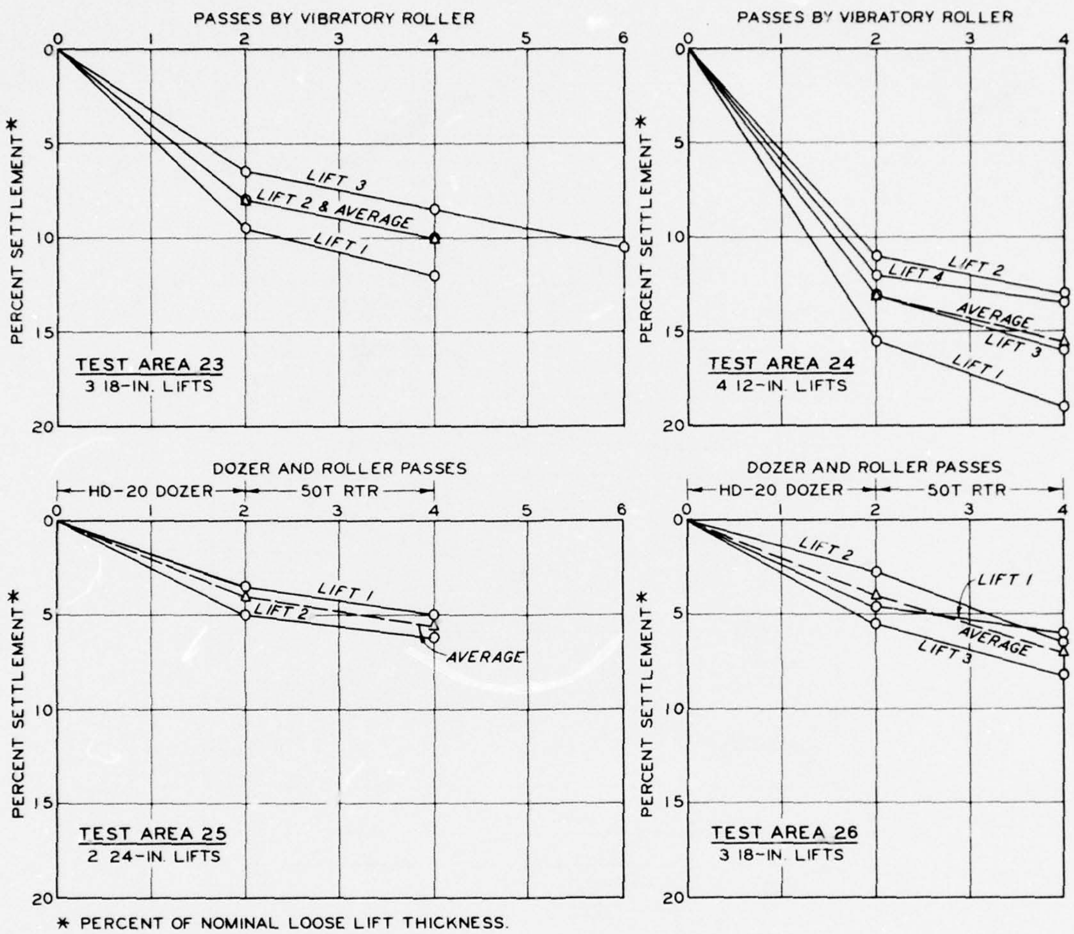
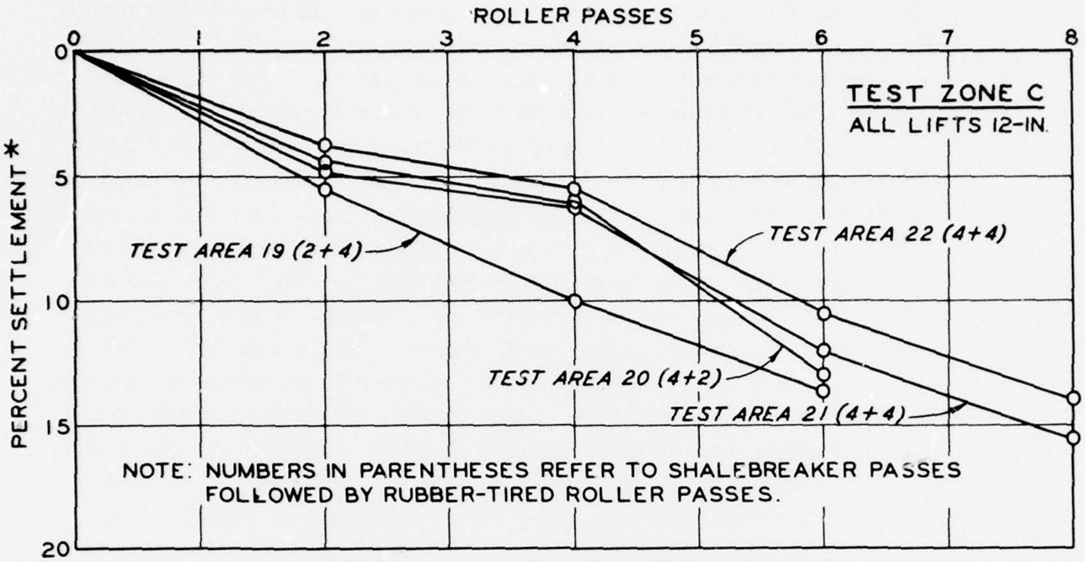


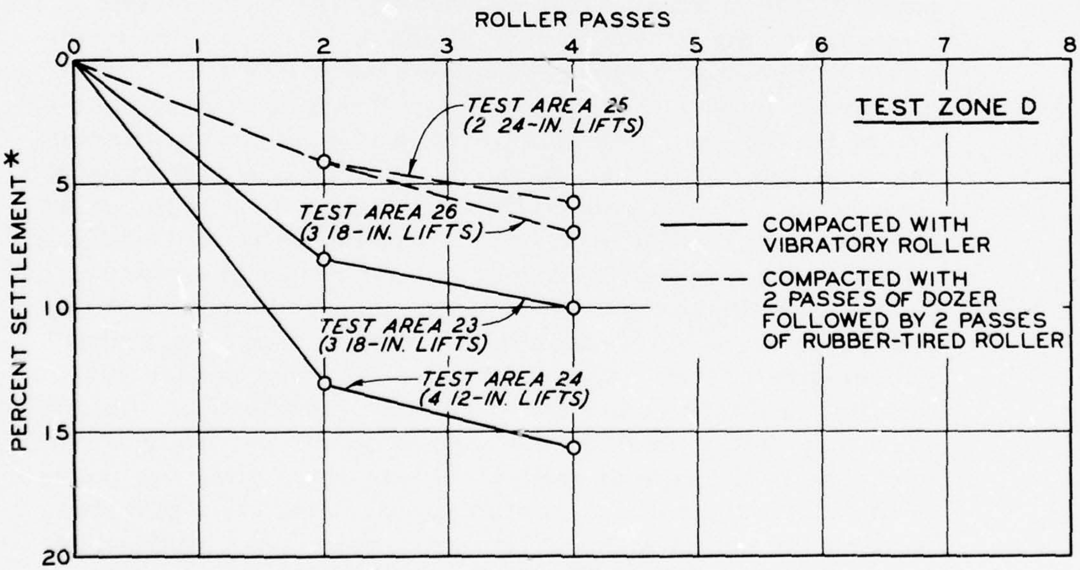
Fig. 9. Percent settlement vs compaction equipment for each lift Test Zone D - relatively unweathered shale Beltzville test fill

settlement, this comparison would have been difficult, as it is obvious that a lift with a loose thickness greater than another lift will undergo more settlement. It should be noted that these settlements are cumulative within each lift (that is, the settlement plotted after four passes, for instance, includes the settlement obtained after the first two passes, etc.). This means the zero settlement reference for each lift is the top of the underlying lift. Therefore, the measured settlement for the lift being compacted includes any additional settlement that occurs in the underlying lift or lifts. The settlement for each interval of rolling (in this case, after every two passes) was obtained by averaging the readings from all 36 points in the grid, except that all points that showed heave and all points that were not in areas of proper initial lift thickness were discarded. The measured settlement of the foundation at completion of the test fill was only 0.03 ft and was considered negligible. Figure 10 contains plots of percent settlement versus number of compaction equipment passes for each test area. These curves were obtained by averaging the percent settlement for each lift in a particular test area. That is, the curves in fig. 10 are averages of the curves in figs. 8 and 9. Since basically the same compaction procedures were used within a particular test area, these curves provide a better means of comparison of the different variables employed. An examination of the upper plot in fig. 10 (partially weathered shale) reveals that the increase in compactive effort from four plus two or two plus four (shalebreaker passes followed by rubber-tired roller passes) to four plus four did not result in a significant increase in the percent settlement. The combination of two passes by the shalebreaker followed by four passes of the rubber-tired roller appeared to give the most reasonable results. For the relatively unweathered shale (fig. 10b), the vibratory roller obtained the best compaction. A significant decrease in the rate of settlement was noted after two passes of the vibratory roller. It also appears from this plot that better compaction with the vibratory roller was attained on the 12-in. lifts than on the 18-in. lifts. There was little difference between the two different lift thicknesses when compacting with the dozer and rubber-tired roller.

d. Inspection trenches. Photographs of inspection trenches in each zone are shown in figs. 11 and 12. These photographs



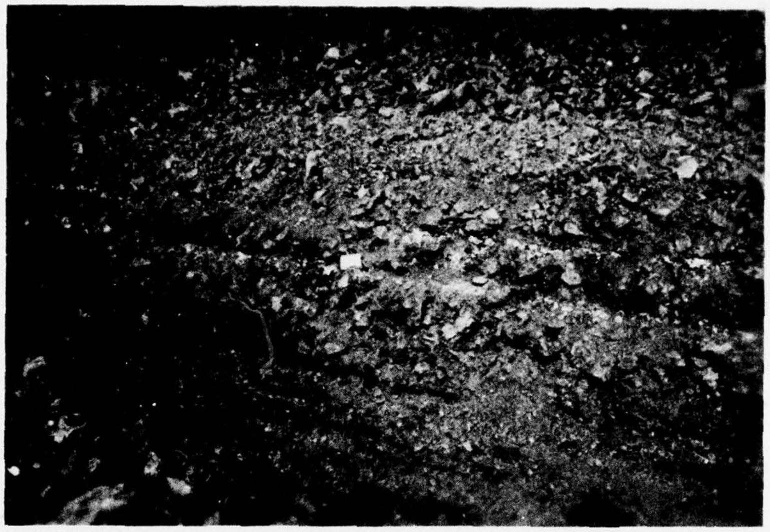
a. PARTIALLY WEATHERED SHALE



b. RELATIVELY UNWEATHERED SHALE

* PERCENT OF NOMINAL LOOSE LIFT THICKNESS.

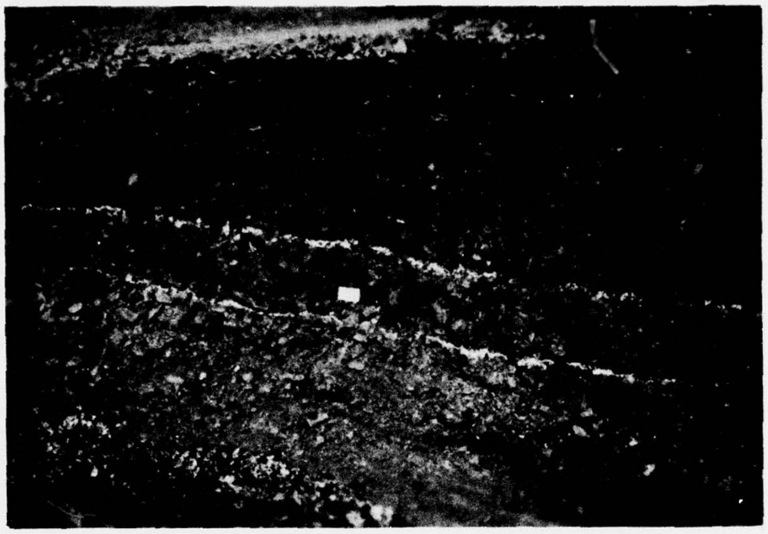
Fig. 10. Percent settlement vs compaction equipment passes for each test area, Beltzville test fill



a. General view

b. Closeup view

Fig. 11. Inspection trench, Zone C, test area 20, partly weathered shale, Beltzville test fill



a. General view

b. Closeup view

Fig. 12. Inspection trench, Zone D, test area 24, relatively unweathered shale, Beltzville test fill

indicate that both materials compacted into a tight, relatively uniform fill. Layering of fines due to surface breakage at the lift boundaries is not evident in the photographs, but was reported from visual observations of the inspection trenches.

Discussion

11. Two comparisons between sand and water as the displacement medium for the large-volume field-density tests showed that use of water gave dry densities 2.7 and 6.6 pcf higher than the sand. Both types of test appear to be equal in the amount of work and care involved in performing the test. Due to the erratic results obtained with the nuclear meter, its use does not appear appropriate on rock fills of this type, although it gave good results in a few cases.

12. The before-and-after gradation curves show that the amount of breakage caused by spreading and compaction was the same for both types of material. The vibratory roller broke down the harder, relatively unweathered shale just as much as the shalebreaker and rubber-tired rollers broke down the partly weathered shale material. However, for the partially weathered shale, the most important factor was to obtain a relatively impervious material to serve as a transition zone between the core and the outer rock shell; permeability was only a secondary consideration. That is, breakage was encouraged rather than discouraged as is often not the case for most rock fills. It would have been very informative had the after-compaction gradation tests been carried out in such a manner as to delineate the distribution of fines with depth in a given lift.

13. The settlement data gave reasonably good indications of the effect of lift thickness and compactive effort. The vibratory roller and 12-in. lifts gave the best compaction for the relatively unweathered shale, as judged by the settlement data.

14. The specifications for construction of Beltzville Dam called for 12-in. loose lifts for both the partly weathered and the relatively

unweathered shale. Compaction for the relatively unweathered shale was by two passes of a 10-ton vibratory roller; while two passes of a shale-breaker followed by four passes of a 50-ton rubber-tired roller was specified for the relatively unweathered shale. The partially weathered shale served as a wide transition (nonfree-draining) zone between an impervious clay core and a thin free-draining outer shell composed of select relatively unweathered shale.

Laurel Dam, Nashville District

General

15. This test fill was planned and constructed in conjunction with the design of Laurel Dam and Reservoir, Cumberland River, Kentucky. The test fill program consisted of two test fills (Nos. 1 and 2); however, test fill No. 1 is not reported herein as it was a very abbreviated investigation and only limited data are available. During construction of test fill No. 1, an abnormally high amount of rock breakage was noticed; this led to the construction of test fill No. 2, which was a more comprehensive testing program. Test fill No. 2 consisted of two types of rock: hard sandstone and soft sandstone. In this report, only the hard sandstone portion will be considered, as the soft sandstone broke down to a tight impervious mass during handling and compaction, and consequently, the procedures employed were similar to those normally employed for an earth test fill. Test fill 2 was constructed between 24 July 1963 and 3 October 1963 under a Government-supervised equipment-rental contract whereby all construction equipment, including operators, fuel, and maintenance, was rented. The supervision for the project was furnished by the Construction Division with technical assistance provided by the Engineering Division, Nashville District. The information contained

herein was largely taken from Design Memorandum No. 3, "Geology and Embankment Design," Part B, January 1964, prepared by the Nashville District.

Rock type

16. The hard sandstone was a light grayish tan, fine to medium rock having pyramidal, cubical, and wedge-like particle shapes. Some samples could be scratched with a knife blade. The larger sizes were fairly resistant to breakage during compaction, but the smaller sizes readily reduced to fine sand under the action of the roller. Maximum rock size allowed in the fill was limited to approximately two-thirds the nominal loose lift thickness used in each section.

Description of test fill 2

17. The test fill consisted of three sections of hard sandstone, as shown in fig. 13. Each section contained a leveling course and two lifts of rock. Section 1 contained two 4-ft lifts; section 2 had two 3-ft lifts; and section 3 was comprised of one 2-ft and one 5-ft lift. Each section was 50 ft square, with a 25-ft transition and turnaround zone provided between sections. The test fill foundation consisted of intact sound rock.

Construction

18. General. Equipment used is described in table 4. Table 5 summarizes construction details. Dump trucks were loaded by a shovel and then hauled the rock from the quarry to the fill, where it was dumped at the leading edge of the advancing lift and spread to the desired thickness with a bulldozer. Spreading was accomplished with as few passes as possible by the dozer in order to minimize surface breakage. After spreading, one pass by the vibratory roller with the vibratory unit turned off was made to even up the fill surface. The vibratory roller, which was used exclusively for compaction, was towed by a D-8 tracked dozer at 1 mph and operated at a frequency of 1500 vpm. No set number of

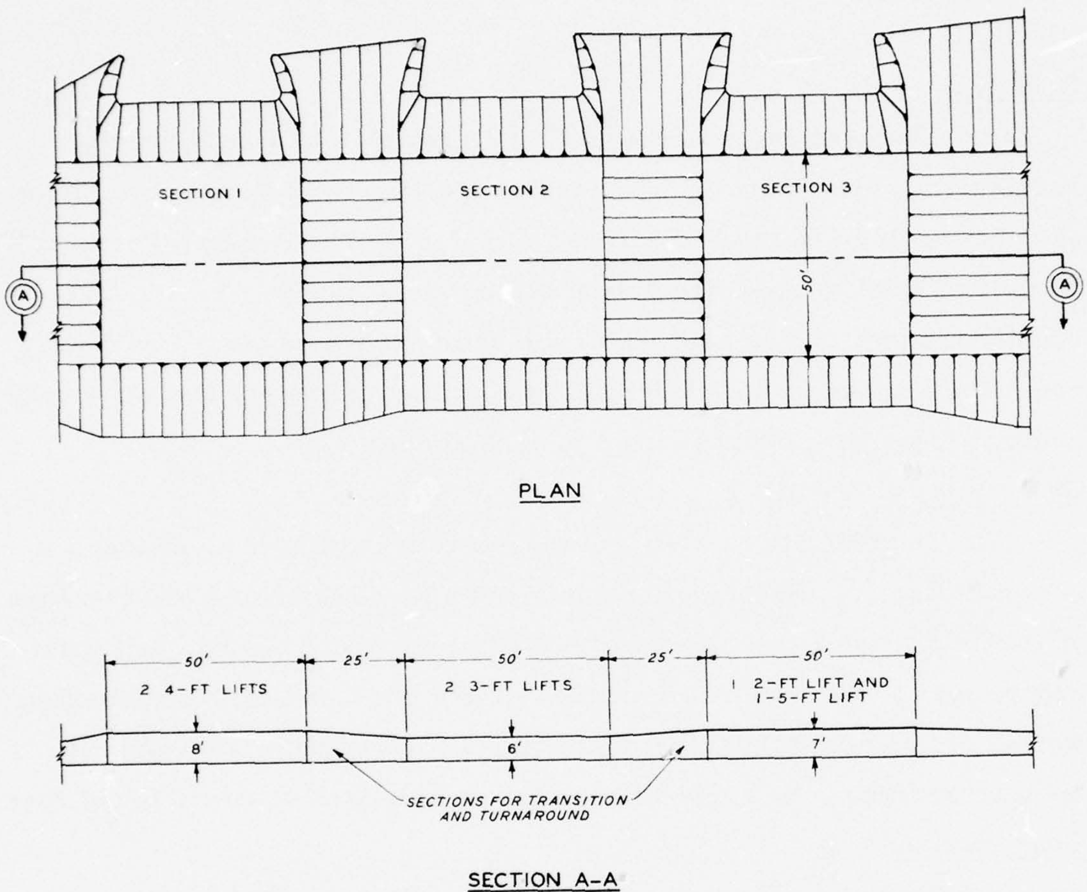


Fig. 13. Laurel test fill plan and profile

Table 4

Construction Equipment, Laurel Test Fill

Item	Description
Shovel	Koehring Model 605, 1-1/2 cu yd, crawler mounted
Truck	Euclid Model 66TD, dump, 27-ton
Tractor	Caterpillar D-8E, crawler, with cable-operated bulldozer blade
Tractor	Caterpillar D-8H, crawler, with hydraulically operated bulldozer blade
Tractor	Caterpillar D-6, crawler, with hydraulically operated bulldozer blade
Roller	Ferguson Model 230, vibratory, 23,600 lb

Note: D-8 dozers were prime movers for vibratory roller.

Table 5

Construction Details, Laurel Test Fill

Section	Lift	Rock Fill*	Nominal Loose Lift Thickness ft	No. Passes	Rolling Pattern
1	1	A	4	10	Forward-backward**
1	2	B	4	12	Forward only
2	1	A	3	10	Forward-backward**
2	2	C	3	8	Forward only
3	1	A	2	8	Forward-backward**
3	2	C	5	8	Forward only

* A - Loaded direct from shot pile in quarry
B - Selective loading to obtain plus 3-in. rock only
C - Loading from dozed rock pile

** Except for last two passes, which were in the forward direction only.

passes were made, but rolling was continued until the measured settlement caused by each two passes was less than 1 percent of the initial lift thickness.

19. Leveling course and first lifts. Before placement of the first lifts, a 3-ft-thick leveling course of hard sandstone was placed and compacted by 10 passes of the vibratory roller to provide a uniform base. All rock for these lifts was loaded directly from the shot pile. The roller was alternately towed forward and backward (towed in the forward direction as is normally done; then, instead of turning, the roller was pushed backwards for the next round in the opposite direction) during compaction of these lifts, except for the final two passes on the 4-ft lift (section 1), which were made by towing in the forward direction only. At the conclusion of the first 3-ft lift (section 2), an effort was made to reduce the number of surface fines by washing the top of the compacted lift with a water hose. This was ineffective.

20. Second lifts. Loading procedures at the quarry were varied for these lifts in an attempt to reduce the amount of fines. The method used for sections 2 and 3 involved (a) pushing the rock up from the floor of the quarry onto the rock pile, (b) pushing the material away from the rock pile (i.e., spreading the rock pile) before loading, (c) holding the shovel bucket 6 in. to a foot off the quarry floor when loading, and (d) shifting the open bucket through the rock pile. A selective loading process was used on the rock for the second 4-ft lift of section 1. This process consisted of selectively loading only the larger size rock (plus 3 in.), in effect simulating the results which would be obtained from a grizzly operation.

21. In an effort to obtain a better distribution of fines throughout these lifts, the vibratory roller was towed in the forward direction only. The construction of the roller was such that the vibratory effect was

greater when towed in the forward direction. It was felt that the increased vibratory effect would cause more of the surface fines to sift down through the layer, thus avoiding a concentration of fines in the upper part of the lift.

Tests and measurements

22. Vertical settlement observations and percolation tests were made to evaluate the effectiveness of the different compaction and loading techniques and the drainage characteristics of the compacted material. An inspection trench was excavated through each section to determine the distribution of fines and to ascertain if satisfactory rock-to-rock contact was obtained throughout the lift. No field density tests or gradation tests were run.

23. Procedures

a. Settlement measurements. The grid layout used to take level readings over each lift is shown in fig. 14. These grid points were established by using a 25-ft-sq template consisting of 3/4-in. pipe around the outer perimeter and diagonally braced with wire. A system of strings stretched from the pipe frame to intersect on 5-ft centers completed the template. The 36 grid points thus located were sprayed with paint for identification. A bench mark was located at either end of the fill so that backsights and foresights could be balanced while readings were being taken on any given section. An initial set of readings was taken to establish actual loose lift thickness. A set of readings was then taken after every two passes of the roller.

b. Percolation tests. These tests, which were run after compaction of the second 3-ft lift in section 2 and the 5-ft lift in section 3, were performed to serve as a rough guide as to whether the lifts could be classified as free draining or not. Standpipes, 3 in. ID by 3 ft high, were placed in the upper portion of the lift in such a manner that little leakage would occur around the bottom. The water level drop in the pipe was then timed, giving a rough indication of the drainage characteristics of the in-place material. The major drawback of this type of test is that it is very susceptible to

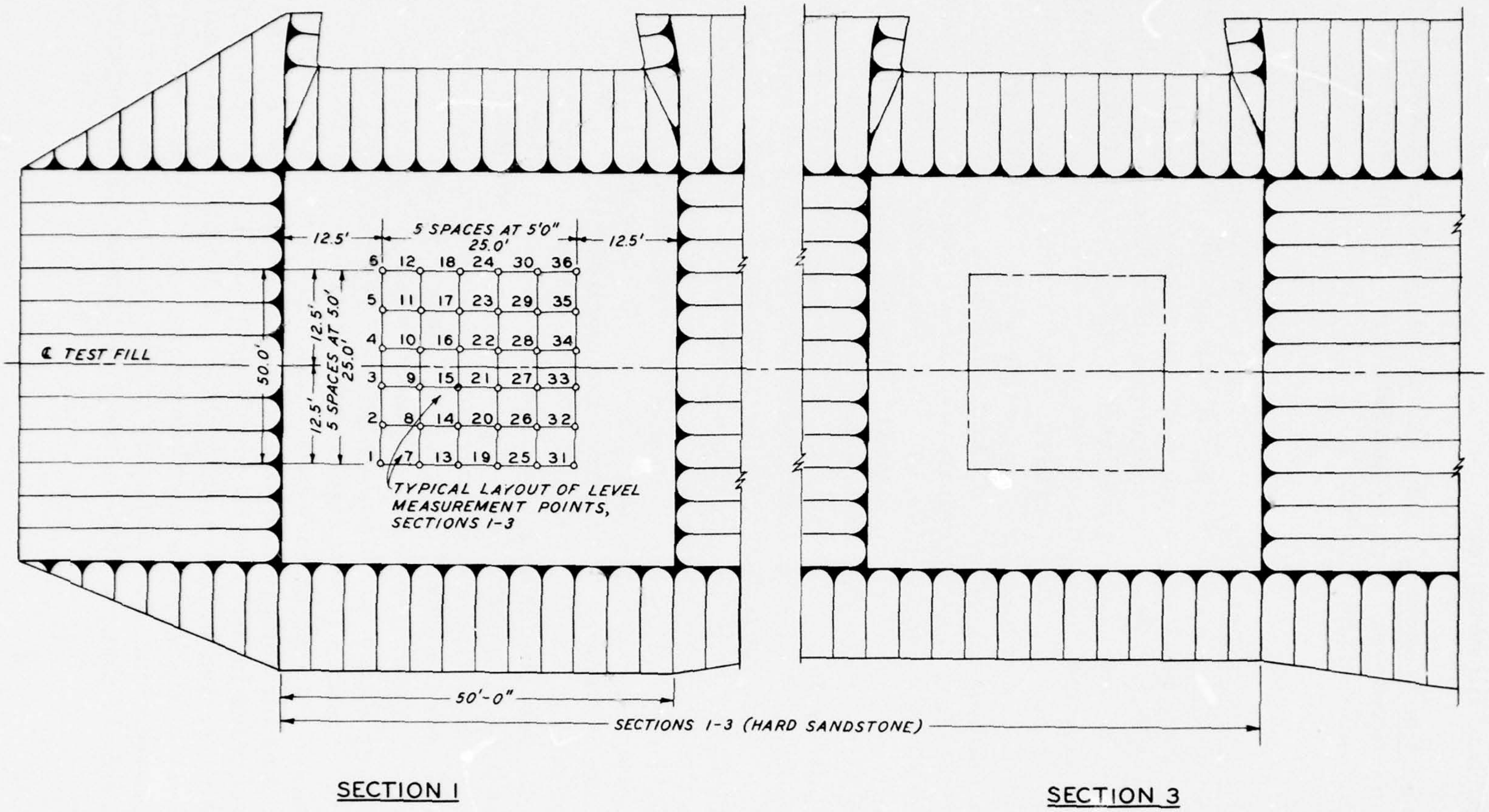


Fig. 14. Layout of leveling grid, Laurel test fill

localized fill conditions.

c. Inspection trenches. The shovel was used to excavate an inspection trench through the full depth of each section after completion of the test fill.

24. Results

a. Settlement measurements. The results of settlement measurements are shown in figs. 15 through 17. Figures 15 and 16 contain plots of percent settlement (percent of actual loose lift thickness) versus number of roller passes, while a plot of settlement in feet as well as percent settlement versus actual loose lift thickness is given in fig. 17. Settlement readings were averaged from readings taken at all 36 grid points as was done previously at the Beltzville project. An examination of figs. 15 and 16 reveals that the second 4-ft lift (for which the previously described selective loading procedure to obtain plus 3 in. rock was used) had the best settlement characteristics; all the other lifts that contained excessive fines showed less settlement except for the 2-ft lift of section 3. The curve for the second 3-ft lift, which was rolled in the forward direction only, indicates slightly more settlement than does the curve for the first 3-ft lift, which was rolled in a forward-backward pattern, indicating that rolling in the forward direction only results in more settlement. However, since only surface settlements were measured, the settlement curve for lift 2 includes any additional settlement that may have occurred in lift 1 from rolling lift 2. The 5-ft lift (fig. 16) did not compact at all well, reaching less than 4 percent settlement after eight passes. It appears from fig. 17 that a 3-ft lift thickness could be chosen as an upper limit in selecting the loose lift thickness for actual construction since the curves for six and eight passes show less settlement in feet occurring in lifts thicker than 3 ft. Also, the percent settlement of lifts thicker than 3 ft was significantly less under six and eight passes than for thinner lifts. (This same type of behavior occurs for some of the data from the Cougar test fill and is discussed in more detail in paragraph 56.) Caution should be exercised in interpreting this kind of data, and it should never be the sole means of selecting variables for prototype construction. For instance, it appears from the lower plot in fig. 17 that the same percent settlement can be obtained

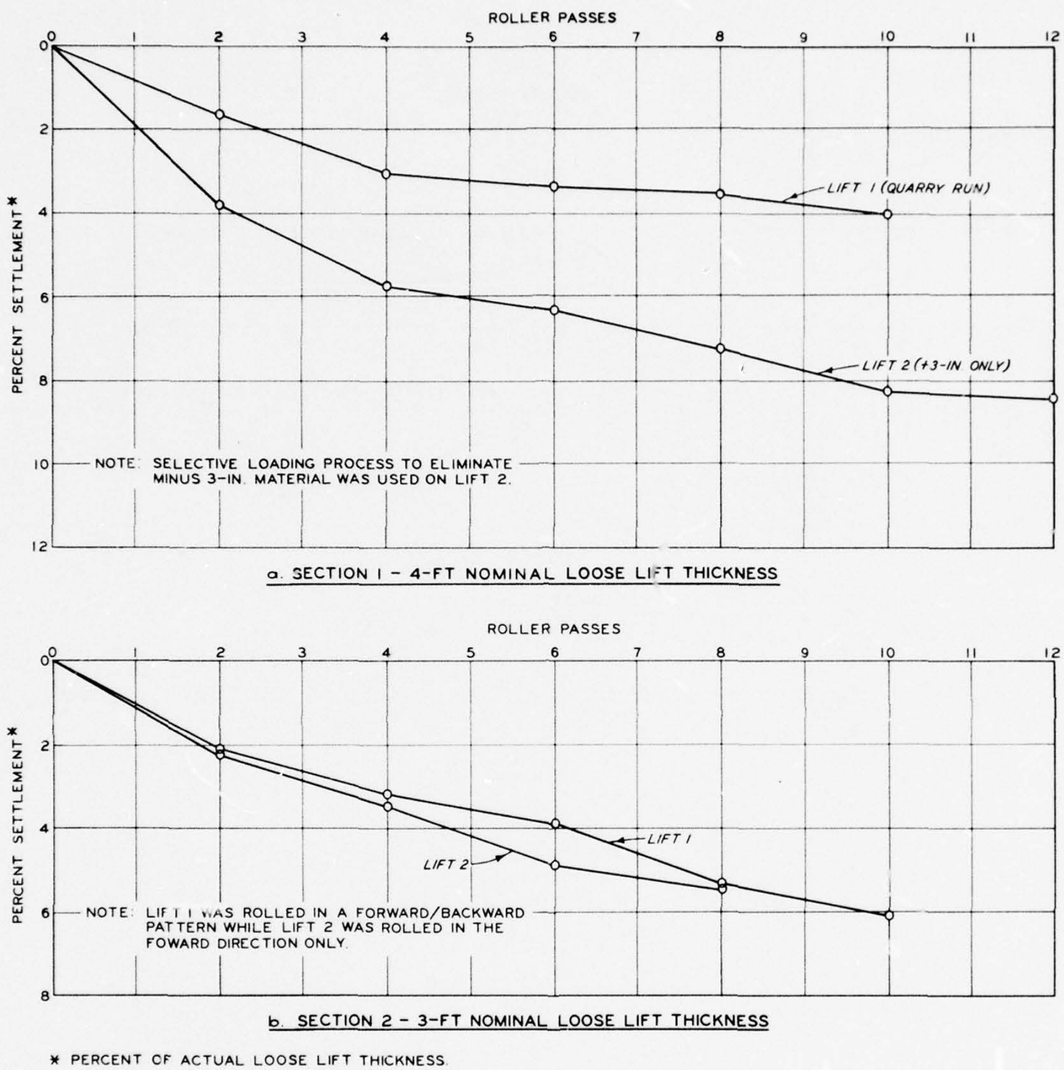
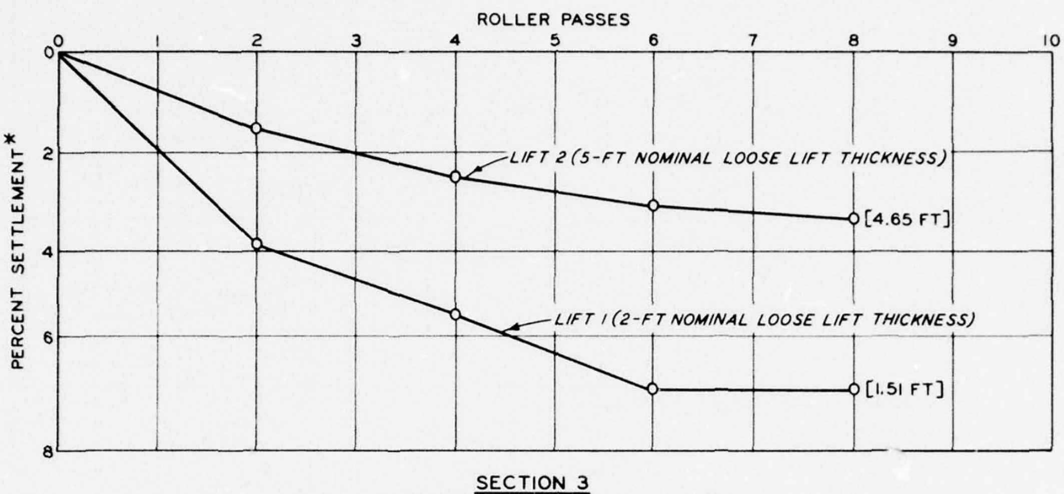


Fig. 15. Percent settlement vs vibratory roller passes sections 1 and 2, Laurel test fill



* PERCENT OF ACTUAL LOOSE LIFT THICKNESS (ACTUAL LOOSE LIFT THICKNESS IS GIVEN IN BRACKETS BESIDE THE APPLICABLE CURVE).

Fig. 16. Percent settlement vs roller passes - section 3, Laurel test fill

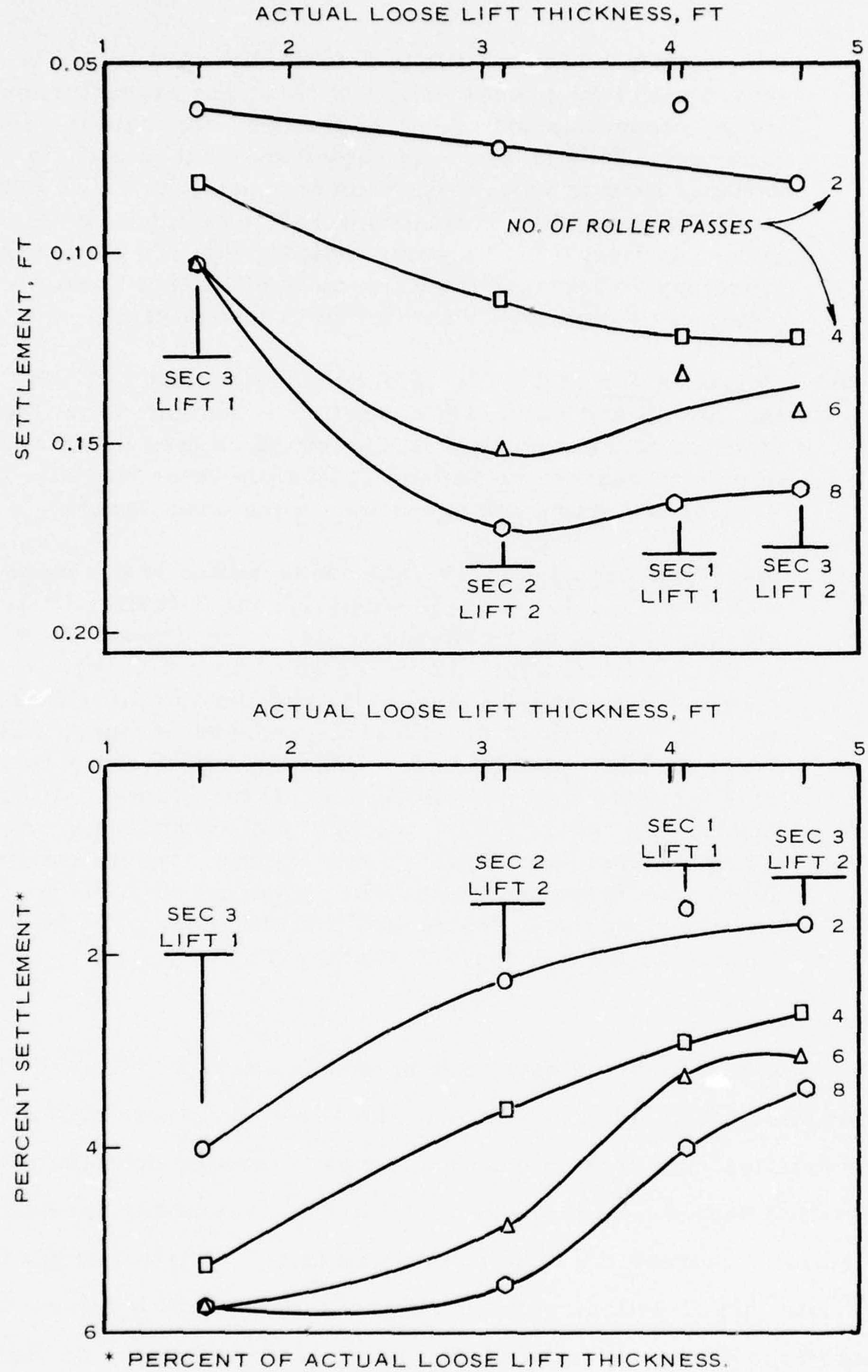


Fig. 17. Percent settlement and settlement in feet versus actual loose life thickness for 2-ft, second 3-ft, first 4-ft, and 5-ft lifts, Laurel test fill

with eight passes on a lift of 5-ft thickness as could be obtained with four passes on a 3-ft lift. However, the uniformity of compaction with depth in the two lifts may be very different. This is why it is very important to include observational results from inspection trenches, and all other available data when evaluating test fill results. Both sets of curves in figs. 15-17 clearly indicate that six passes of the vibratory roller accounted for most of the total settlement obtained by eight passes at lift thicknesses of 3 ft or less.

b. Percolation tests. Tests in both the second 3-ft lift (section 2) and the 5-ft lift (section 3) indicated that the in-place material could not be classified as free draining. Attempts to reduce the amount of fines in these two lifts by varying the quarrying operations were unsuccessful.

c. Inspection trenches. Visual observations of the inspection trenches revealed the following: (1) the 2-ft lift (lift 1, section 3) had practically no voids, being choked throughout by fines; (2) the upper 12 in. or so of both 3-ft lifts (section 2), the first 4-ft lift (section 1), and the 5-ft lift (lift 2, section 3) contained considerable amounts of fines, while the lower portions contained few fines but satisfactory rock-to-rock contact was in evidence; and (3) the second 4-ft lift (section 1), where plus 3-in. material was placed, revealed a much better distribution of fines throughout its entire depth. An inspection trench in the second 4-ft lift is compared to one in the second 3-ft lift in fig. 18; the greater number of fines in the 3-ft lift is evident.

Discussion

25. It was recognized early during construction of this test fill that too many fines were being produced at the surface. It was also observed that the smaller size rock (minus 3 in.) was the major contributor to this situation because of its ready reduction to fines under spreading and roller action, whereas the larger rock was fairly resistant to breakage. To alleviate this situation, several different methods of loading and working in the quarry were tried, but only selective loading of the larger size rock was successful. This would seem to indicate a necessity for

a. Second 3-ft lift

b. Second 4-ft lift

Fig. 18. Inspection trenches, Laurel test fill

use of a grizzly on the prototype fill.

26. These initial observations were verified by the inspection trenches which, upon inspection, revealed a layer of about 1 ft of tightly packed fines in the upper part of all lifts except in the second 4-ft lift, for which the selective loading procedure was used. This choked condition in the upper part of the lifts reduced the permeability to such an extent that the in-place material could not be classified as free draining. Figure 19 compares the surfaces of two of the lifts with excess fines to the surface of the second 4-ft lift (plus 3-in. material only) after the same number of passes.

27. The effect on settlement of compacting only the larger size material must also be evaluated. It appears from the settlement data and from inspection trenches that the second 4-ft lift had the best settlement characteristics of all the lifts placed. It is felt that this was primarily due to the production of fewer surface fines, which allowed those fines that were produced to sift uniformly throughout the mass. These results also lend credibility to the assumption that a dense layer of surface fines bridges or arches over the underlying material and not only prevents a uniform distribution of fines but also dampens the vibratory effect on the underlying material as well.

28. The equipment-rental type of contract used for this project appears to have allowed maximum flexibility for construction of the test fill.

29. Specifications for Laurel Dam called for the hard sandstone to comprise the outer free-draining rock shell. It was placed in 3-ft loose lifts and compacted by six passes of a 10-ton vibratory roller. A screening operation was specified for this rock to limit its minimum size to about 3 in. It was further specified that only rubber-tired equipment be allowed on the fill in this zone.

5-ft lift (quarry run)

Second 4-ft lift (plus 3 in. only)

a. After four passes

Second 3-ft lift (quarry run)

Second 4-ft lift (plus 3 in. only)

b. After six passes

Fig. 19. Lift surfaces, Laurel test fill

Gathright Dam, Norfolk District

General

30. This test fill program was conducted as a part of the field investigation for the design of Gathright Dam and Reservoir, Jackson River, Virginia. The purpose of the program was to determine (a) the suitability of available rock for use in the embankment, (b) the proper gradation of rock, and (c) the optimum number of roller passes and lift thickness. The test fill was designed by the Mobile District with supervision of construction and field testing accomplished by personnel from both the Mobile and Norfolk Districts. Most of the information in this summary was taken from unpublished notes and data on the test fill prepared by Mobile District.

Rock type

31. The rock used in the test fill consisted of moderately hard to hard, medium, gray, sandy limestone with shale laminae and calcite layers. Particle shape after blasting was classified as angular. Both grizzled and quarry-run gradations were used in the fill, as will be discussed in more detail later. There were no prerolling gradations, but a single gradation curve after rolling is shown in fig. 20. Maximum allowable rock size was approximately two-thirds of the nominal loose lift thickness.

Description of test fill

32. A plan of the test fill is shown in fig. 21, and three sections through the fill are shown in fig. 22. Two zones were specified, one having five 18-in. lifts and the other four 24-in. lifts. The first three 18-in. lifts and all of the 24-in. lifts were composed of rock retained on a grizzly having 2.5-in. openings. The last two 18-in. lifts were composed of quarry-run rock. The test lanes in each zone were 50 ft long

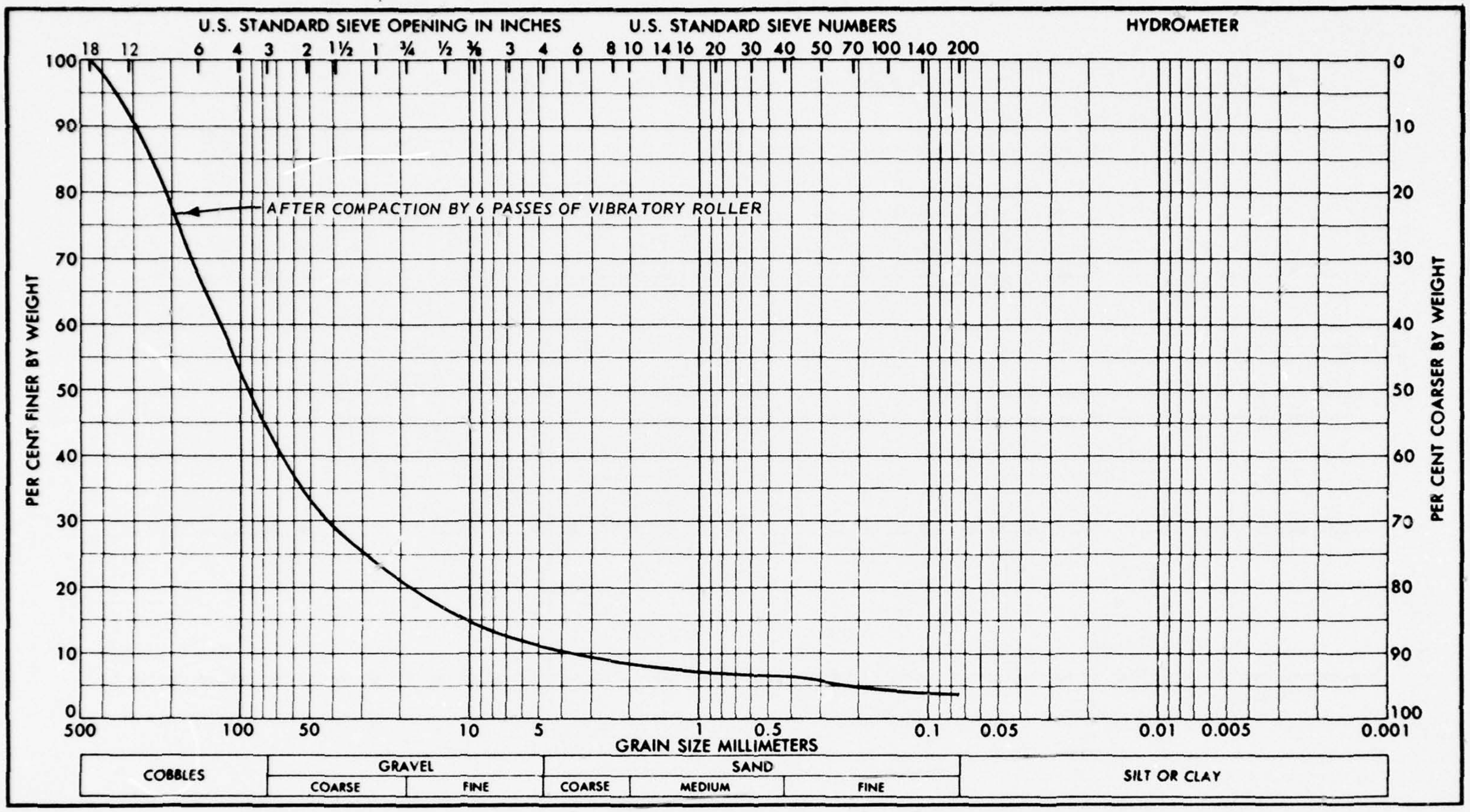


Fig. 20. After compaction gradation, 18-in. lift, quarry-run material, Gathright test fill

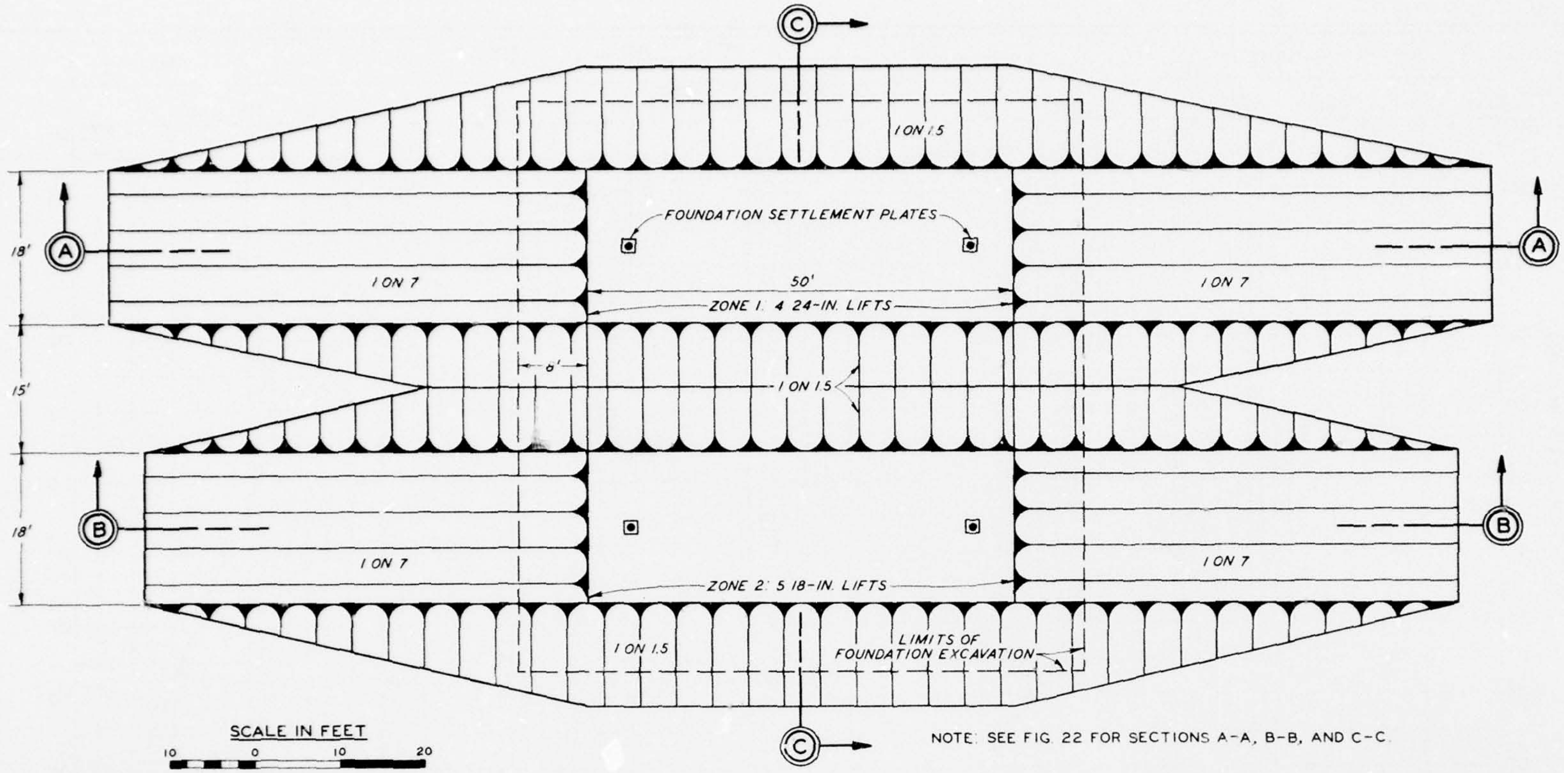


Fig. 21. Plan, Gathright test fill

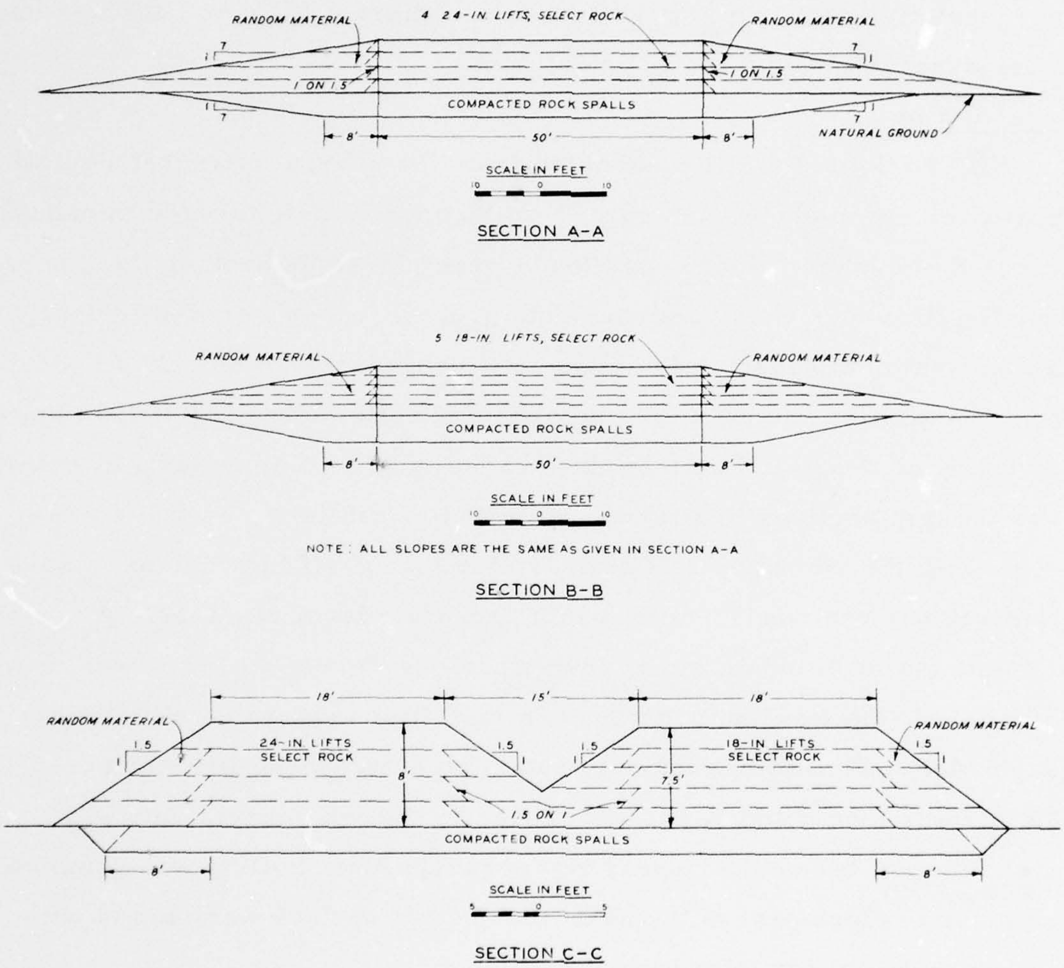


Fig. 22. Fill sections, Gathright test fill

and 18 ft wide. Access was provided to the fill from both ends by ramps sloped at 1V on 7H. Turnaround areas for the equipment were provided beyond the test fill limits. To provide a firm base, approximately 3 ft of overburden was removed and replaced with compacted spalls. Settlement plates were installed at locations shown in fig. 21 in order to ensure representative data from the settlement readings. Random rock was used to construct the ramps and fill side slopes.

Construction

33. Rock for the fill was hauled from the quarry in trucks, dumped approximately in place, and spread to the approximate desired thickness by a D-8 bulldozer. Successive loads were placed by backing the trucks over the lift under construction. This procedure was intended to duplicate normal dam construction placement procedures as closely as possible. Compaction was by a 10-ton vibratory roller vibrating at 1375 vpm and towed at 1 to 1-1/4 mph by the D-8 bulldozer. The roller was towed over the test sections in alternate directions. Initially, each lift was rolled with the vibratory unit of the roller turned off in order to achieve a smooth surface conducive to making accurate level readings. A system of points (grid) to locate where level readings were to be taken was then established (the leveling procedure is explained in more detail in paragraph 34b), and readings taken to establish actual loose lift thickness. The lift was then subjected to six passes by the vibratory roller with level readings taken after every two passes. After rolling was complete and prior to placement of the next lift, the lift surface was spread with marker material for later identification.

Tests and measurements

34. Procedures

a. Field density tests. Two density tests were performed after construction was complete. One test was in lift 5 of the 18-in. zone (quarry-run material) and the other was in

lift 4 of the 24-in. zone (grizzled material). Both tests were performed by the large-scale water-displacement method. In this method, a 3- by 3-ft wood frame was laid down on the surface, and a hole in the fill approximately 12 to 15 in. deep was excavated within the confines of the template. The excavated material was weighed, and the volume of the hole was determined by carefully fitting a thin plastic liner in the hole and measuring the amount of water required to fill the hole.

b. Gradation test. Material for the gradation test shown in fig. 20 was shipped to the South Atlantic Division Laboratory for testing where regular screens were available up through the 6-in. size. All larger rock was individually sized with metal frames that varied in size up to the maximum size rock.

c. Settlement measurements. The grid established to make settlement readings is shown in fig. 23. As mentioned earlier, these points were located on the surface of each lift after the roller had made one pass with the vibratory unit off. Spray paint was used to mark the points to facilitate identification throughout the rolling process. Level readings were taken initially to establish the actual loose lift thickness and subsequently after every two passes of the roller to measure the settlement. Readings were also taken on the riser pipes brought up through the fill from the foundation settlement plates at the completion of each lift.

d. Inspection trenches. At the conclusion of construction, one side of each zone was cut away by the bulldozer, and a cross section suitable for visual examination was exposed.

35. Results

a. Field density tests. The results of the two field density tests were as follows:

(1) Lift 5, 18-in. material (quarry run), 142 pcf.

(2) Lift 4, 24-in. material (grizzled), 118 pcf.

b. Settlement measurements. A plot of the foundation settlement is shown in fig. 24. Figure 25 shows the percent settlement of each lift versus the number of roller passes.

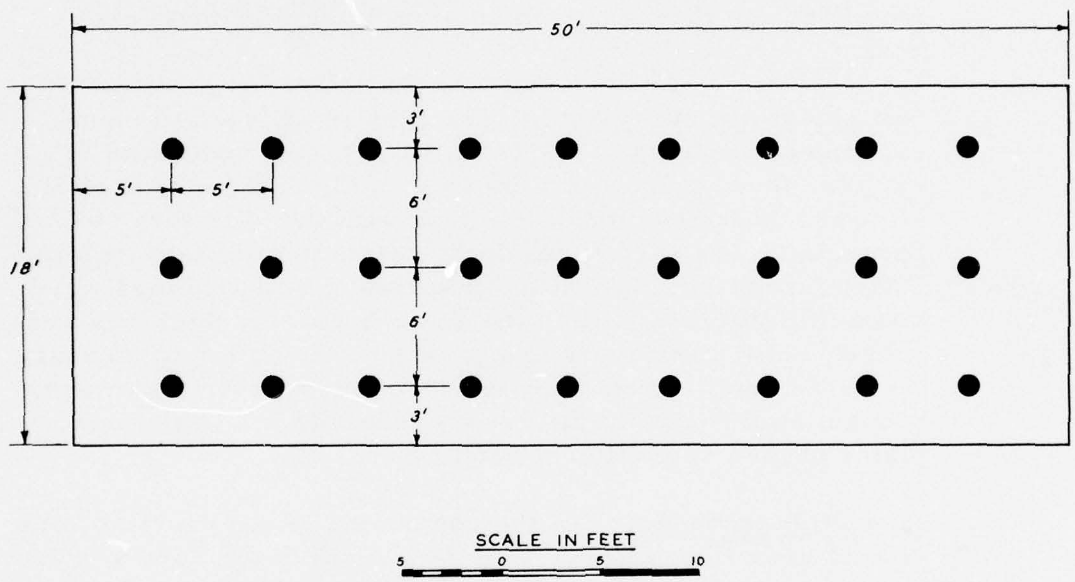


Fig. 23. Settlement reading locations, Gathright test fill

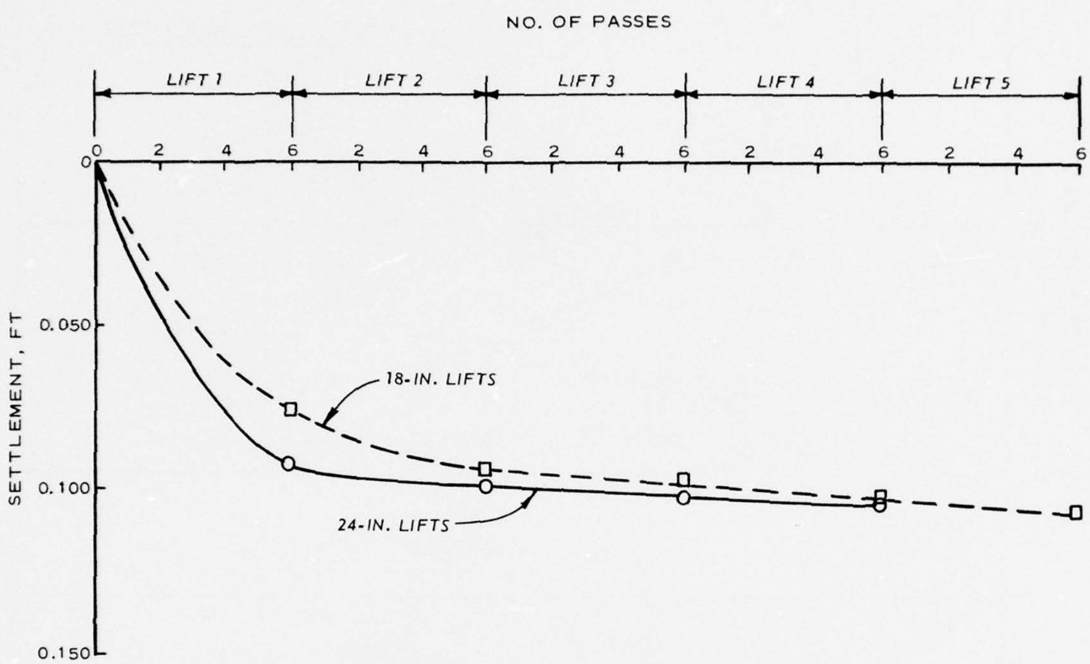


Fig. 24. Foundation settlement, Gathright test fill

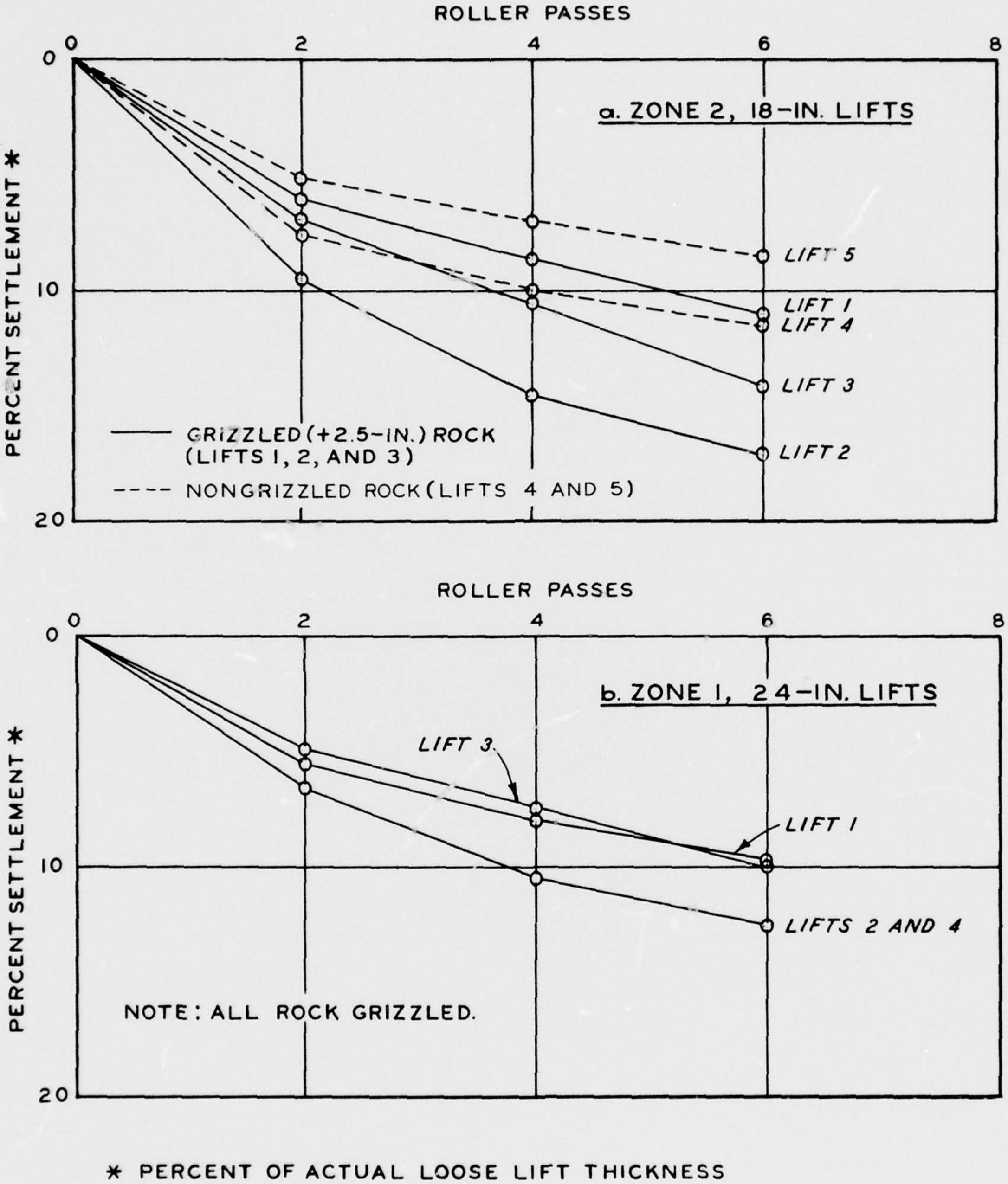


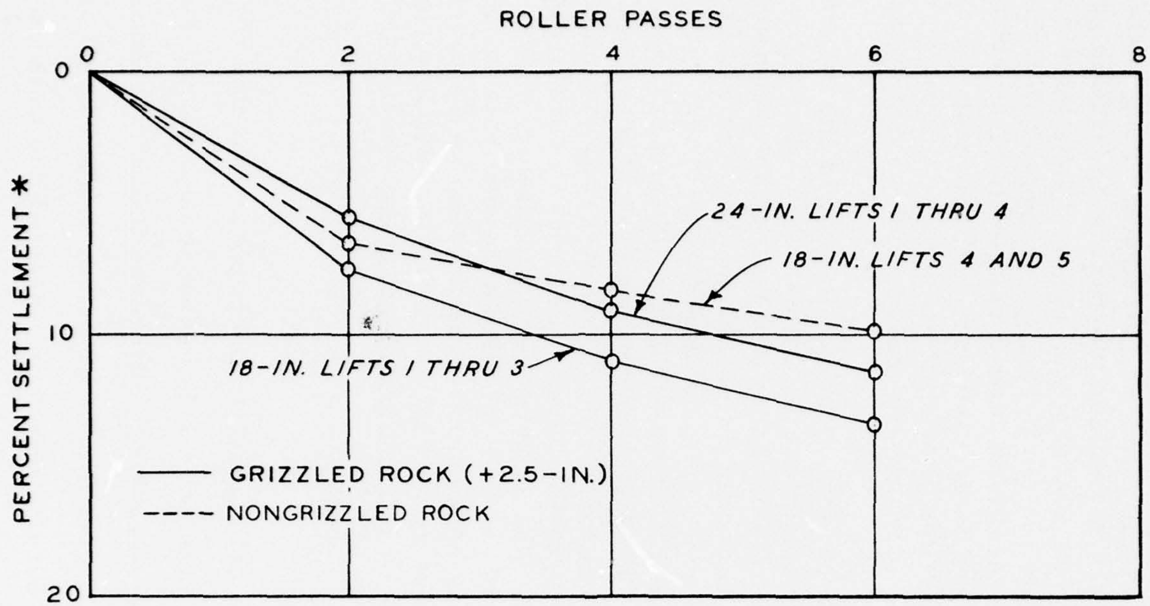
Fig. 25. Percent settlement vs roller passes for each lift, Gathright test fill

These curves were obtained by averaging the level readings from all points in the grid, subtracting the foundation settlement, and dividing by the actual loose lift thickness. The curves shown in fig. 26 are averages from the curves in fig. 25 and are presented to facilitate comparison between the 18- and 24-in. lifts and the grizzled and nongrizzled rock. As can be seen in fig. 24, most of the foundation settlement occurred during placement of the first lifts in both zones. From the data plotted in fig. 26, it appears that the use of 18-in. lifts resulted in better compaction than did the use of 24-in. lifts. The grizzled material appeared to exhibit better compaction characteristics than the quarry-run material for four and six passes of the vibratory roller.

c. Inspection trenches. An examination of the inspection trenches revealed that the uniformity of compaction was good; i.e., no pockets of segregated material were in evidence. There was a marked difference in the appearance of the quarry-run rock and the grizzled rock in that voids so evident in the grizzled rock were filled with fines in the quarry-run material. The degree of compactness of these fines was not physically measured, but it was observed that no fines fell out when the vertical side was formed in cutting the inspection trench. The grizzled rock appeared to be well compacted in both the 18- and 24-in. lifts with good rock-to-rock contact. Good bonding between lifts was attained as was evidenced by penetration of material from overlying lifts into underlying lifts. Very little evidence of breakage was found except at the lift surfaces.

Discussion

36. From the data collected, it would appear that this rock is an excellent construction material and is usable in both quarry-run and grizzled forms. There was a large difference in the compacted densities of the two gradations (118 pcf for the grizzled versus 142 pcf for the quarry-run material), but this was to be expected due to the amount of fines present in the quarry-run material in the voids between the larger rock. It should be emphasized that in the case of compacted rock fills, the fact that a material of one gradation attains a higher density than one



* PERCENT OF ACTUAL LOOSE LIFT THICKNESS

Fig. 26. Percent settlement versus roller passes (average of curves given in fig. 25), Gathright test fill

of different gradation does not necessarily indicate a better fill. The degree of interlocking of the rock and rock-to-rock contact must also be considered as an indicator, not just density alone.

37. It would also appear that four passes of the vibratory roller gave reasonable compaction, although none of the settlement curves for the grizzled material leveled off after six passes.

38. Breakage of this rock appeared to be negligible as was evidenced from inspection of the fill surface during rolling and the sides of the inspection trenches after rolling. This rock appears to be an excellent material for compaction by a vibratory roller.

39. Conclusions reached by the Mobile District were as follows:

- a. The rock zone of the embankment should be placed in 24-in. loose lifts.
- b. Four passes of a 10-ton-class vibratory roller should be used to compact each lift.
- c. The grizzled and quarry-run rock are both good construction materials and the decision as to which should be used (i.e., zonation) should be determined by a cost analysis.

Cougar Dam, Portland District

General

40. Cougar Dam test fills were constructed as part of the rock-fill embankment during construction of the dam and were the first experiments conducted in the United States for compacting rock fill for earth and rock-fill dams by vibratory rolling. The design, supervision of construction, and all field tests and measurements were performed by the Portland District. The information in this summary was taken from a paper by Bertram* and information supplied by the Portland District.

* Bertram, G. E., "Rockfill Compaction by Vibratory Rollers," Proceedings, Second Pan-American Conference on Soil Mechanics and Foundation Engineering, vol I, 1963, pp 441-445.

41. The test program consisted of three different series of fills. The first series was initiated when it became apparent that two passes of a 50-ton rubber-tired roller would not produce any noticeable improvement in the appearance of the layer over that obtained with the hauling and spreading equipment. Three different types of rollers were evaluated in this first series. A second series was deemed prudent in order to evaluate a new 10-ton vibratory roller which had just become available. The purpose of the third series was to obtain a more accurate definition of the relationship between actual settlement and layer thickness for rock fill having a definite maximum size. The construction schedules for the three series of fills are summarized in table 6.

Rock type

42. The rock used in these test fills ranged from a black, fine-grained (sometimes glossy) basalt to a gray-brown, medium-grained vesicular basalt. The fine-grained and glossy basalt generally broke to relatively small angular shapes with considerable fines produced by the blasting and roller action. The medium-grained basalt broke to larger angular and blocky shapes with less fines than the fine-grained basalt. Test fills of the first series were composed primarily of the fine-grained rock, while those of the second and third series consisted mostly of medium-grained rock. Rock for the first two series was quarry run with maximum sizes limited to about two-thirds the nominal loose lift thickness; in the third series, rock was screened through a 12-in. grizzly to restrict the maximum size. This processed material was uniformly graded from a maximum size of 12 in. to about 2 percent passing the No. 4 sieve.

Test fill layout and construction

43. The test fill surface dimensions ranged in widths from 35 to 50 ft and lengths from 80 to 125 ft. Access was provided at each end of

Table 6

Construction Schedule, Cougar Test Fill

Test Series	Rock Type and Gradation	No. of Lifts	Nominal Loose Lift Thickness in.	Roller*	No. of Roller Passes
1	Fine-grained, quarry-run				
	basalt	6	18	50-ton rubber-tired	4
	Maximum size - 2/3 loose lift	6	18	5-ton vibratory	4
	thickness	6	18	Experimental impact	4
2	Medium-grained, quarry-run	1	18	5-ton vibratory	6
	basalt	1	24	5-ton vibratory	6
	Maximum size - 2/3 loose lift	1	36	5-ton vibratory	6
	thickness	1	18	10-ton vibratory	6
		1	24	10-ton vibratory	6
		1	36	10-ton vibratory	6
		1	18	5-ton vibratory followed	12**
		1	24	by 10-ton vibratory	12**
		1	36		12**
3	Medium-grained basalt,	1	18	10-ton vibratory	6
	grizzled (+12 in. removed)	1	24	10-ton vibratory	6
	12-in. maximum size	1	30	10-ton vibratory	6
		1	36	10-ton vibratory	6

* Full descriptions of rollers are given in paragraphs 49 and 52.
** 6 passes of 5-ton vibratory followed by 6 passes of 10-ton vibratory.

the fills by ramps sloped 1V on 5H. Turnaround areas for the equipment were located beyond the limits of the test sections.

44. Trucks were used to haul the rock from the quarry to the fill, with each load dumped approximately in place after backing over the lift being placed. Spreading was done by a D-8 bulldozer having a total weight of 55,000 lb. Each lift in the first series received four passes, and each lift in the second and third series received six passes of the particular roller being used. The rollers were towed over the test sections in alternate directions. In general, placement procedures duplicated those in use for the normal dam construction routine. A more detailed discussion of the construction is contained in later paragraphs under each particular test series heading.

Measurement of compaction obtained

45. To determine compaction obtained by the different rollers being tested, large-scale water-volume density tests were made in the first series of fills, and settlement measurements were made in the second and third series.

46. Density tests. Nine density tests were performed at the completion of the first series, three for each type roller used. Of the three, one was performed at the completed fill surface, one at a depth of 3 ft and one at 6 ft. The latter two tests were performed after excavating through the fill down to the desired elevation. Procedures for performing the density tests were similar to those described previously for other test fills. A 6-ft-diam metal ring was used as the surface template, and the volume of the hole dug in each test was approximately 1 cu yd. The volume of the hole was determined by measuring the water needed to fill the hole after it was lined with a thin plastic membrane to prevent leakage.

47. Settlement measurements. After each lift was uniformly

spread to the approximate desired thickness, a grid of settlement reading points was established on the surface of the fill from reference markers located beyond the test fill limits. The lift surface at each grid point was marked with a splash of paint. Initial level readings were then taken to establish the actual loose lift thickness; readings were taken after every two passes of the roller thereafter.

48. In order that the readings taken would reflect an average elevation, a 1-ft-sq metal plate was placed over the point. The level rod was placed on a raised button in the middle of the plate. A handle made from a steel rod was attached to the plate, which allowed the plate to be moved about slightly to obtain firm seating on the rock. This apparatus is pictured in fig. 27.

Test fill series No. 1

49. This test fill was constructed in 1961 for the purpose of comparing the compaction obtained with a 50-ton rubber-tired (pneumatic) roller, an experimental impact roller, and a 5-ton vibratory roller. The performance of the experimental impact roller is not reported herein because its performance was substandard, resulting in it being ruled out as an effective piece of compaction equipment. The tabulation below gives some specifications on the other two rollers used:

Roller	Prime Mover	Speed mph	Frequency of Vibration vpm
Tampo, 50-ton pneumatic	D-9 tracked bulldozer	3	--
Bros, VP-9D, 5-ton Vibropactor (vibratory)	Euclid 15-ton rubber-tired tractor	1.5 - 2	1400

50. The test fill was so arranged that there were three lanes (one for each type roller) with six 18-in. lifts per lane, each compacted by

Fig. 27. Apparatus for taking level readings, Cougar test fill

four roller passes. No settlement data were taken, but density tests were made as described previously. Results of these density tests are tabulated below:

Depth to top of Density Test Hole	Dry Density, pcf			
	50-ton Rubber-Tired		5-ton Vibratory	
Surface	110		117	
3 ft	121	119.5	115	120.5
6 ft	118		126	
Average	116		119	

51. These results tend to indicate that the greater amount of compaction was obtained with the 5-ton vibratory roller, but average results below a depth of 3 ft were about the same. Visual observations made during compaction indicated that the vibratory roller was producing the best fill. This resulted in the 5-ton vibratory roller replacing the 50-ton rubber-tired roller for actual construction.

Test fill series No. 2

52. A new Bros Model VP-20D, 10-ton vibratory roller was made available in the spring of 1962, and it was decided to conduct a second series of test fills in order to compare the effectiveness of the new roller with that of the 5-ton vibratory roller, which, as mentioned previously, was being used on the main embankment. Both rollers operated at a frequency of vibration of approximately 1400 vpm and were towed at speeds between 1.5 and 2 mph by a rubber-tired tractor weighing approximately 15 tons. Three test zones were utilized, each consisting of single lifts of 18-, 24-, and 36-in. nominal thicknesses, respectively. Two test lanes were used per zone, one for the 5-ton and one for the 10-ton vibratory roller. Settlement readings were taken as previously described. In the lanes for the 10-ton roller, each lift was compacted by six passes; in the lanes for the 5-ton roller, the lifts were

compacted by six passes. Then, since the 5-ton vibratory roller was being used on the construction of the dam, it was decided to determine the effect of six passes of the 10-ton roller on fill previously compacted with six passes of the 5-ton roller.

53. Results of the second series are shown in figs. 28 and 29. Figure 28 shows percent settlement versus actual loose lift thickness for both the 5- and 10-ton rollers. It clearly indicates the superior compaction obtained by the heavier roller. Figure 29 is also a plot of percent settlement versus loose lift thickness, but compares the effects of the 10-ton roller alone and that of the 10-ton roller plus the percent settlement produced by previous compaction by six passes of the 5-ton roller. These data indicate that the 10-ton roller is more effective when used alone; i.e., more compaction was obtained with six passes of the 10-ton roller alone than was obtained with six passes of the 5-ton roller followed by six passes of the 10-ton roller. In his paper, Bertram attributed this lack of effectiveness of the superimposed rolling to an interlocking of the rock produced by the 5-ton roller which the heavier roller could not release. Also to be considered is the fact that the 5-ton roller produced fines on the surface of the fill and these fines, in addition to those inherently present, might form a tight compact mass in the upper part of the lift which could dampen any subsequent vibration to the lower part of the lift.

Test fill series No. 3

54. A third and final series of test fills were conducted a few months after the second series in order to obtain more accurate definition of the relationship between settlement and loose lift thickness. It was believed that the variations in the preceding data might have been caused by random large rocks in the separate lifts as the top size placed depended on the thickness of the lift (i.e., the thicker the lift, the larger the

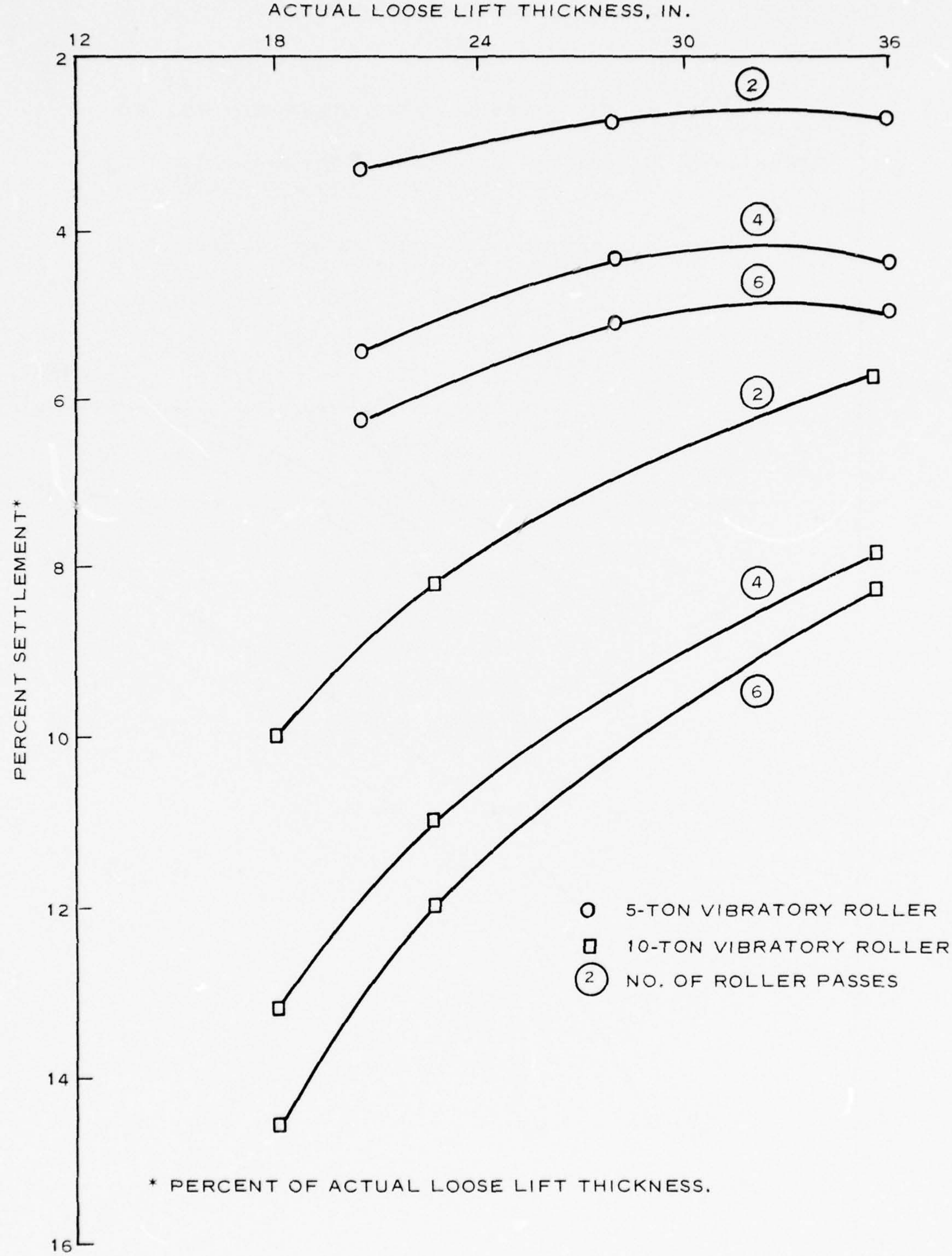


Fig. 28. Percent settlement vs actual loose lift thickness, test fill series 2, Cougar test fill

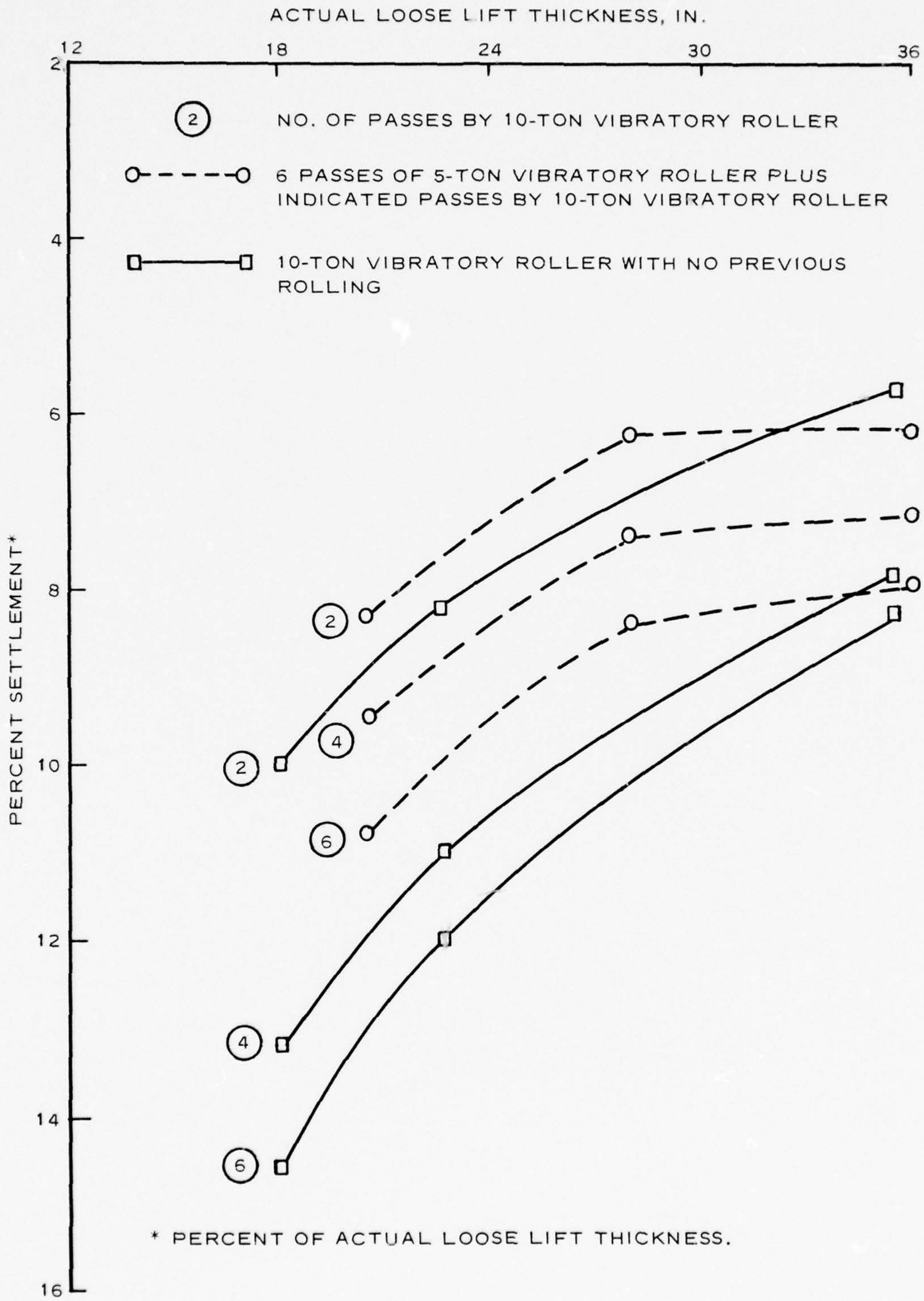


Fig. 29. Percent settlement vs actual loose lift thickness, test fill series 2, Cougar test fill

maximum size rock allowed). It was decided therefore to limit the maximum rock size in this series to 12 in. for all lifts. This was accomplished by dumping the rock over 12-in. horizontal splitter bars. The resulting material was uniformly graded, having a maximum size of 12 in. with only 2 percent passing the No. 4 sieve. Nominal lift thicknesses of 18, 24, 30, and 36 in. were used. The 10-ton vibratory roller was used exclusively, and a total of six passes were made over each lift with settlement readings made after each two passes as before.

55. Results from this series of test fills are shown in fig. 30, where both percent settlement (percent of actual loose lift thickness) and actual settlement in feet are plotted versus loose lift thickness. These data are somewhat irregular, but they do illustrate that the maximum rock size does influence compaction. This is evidenced by the difference in the shape of the percent settlement curves when compared to those in figs. 28 and 29 where the maximum rock size was governed by the lift thickness (i.e., two-thirds the loose lift thickness) rather than being a set value (12 in.) for all lift thicknesses as was done in this third series. The percent settlement curves show an overall decrease with increasing lift thickness but do undergo a change in slope at about 32-in. thickness, indicating that the rate of percent settlement decreased much faster with increasing lift thickness beyond that point.

56. Actual settlement was also plotted versus loose lift thickness to see if it would yield any more information toward the purpose of this test series, which was to more accurately develop the relationship between settlement and loose lift thickness. As can be seen from these curves in fig. 30, the maximum actual settlement is reached at about the 32-in. loose lift thickness. However, since these curves are for actual settlement and not percent settlement, this thickness cannot be considered an optimum loose lift thickness, but does seem to be a

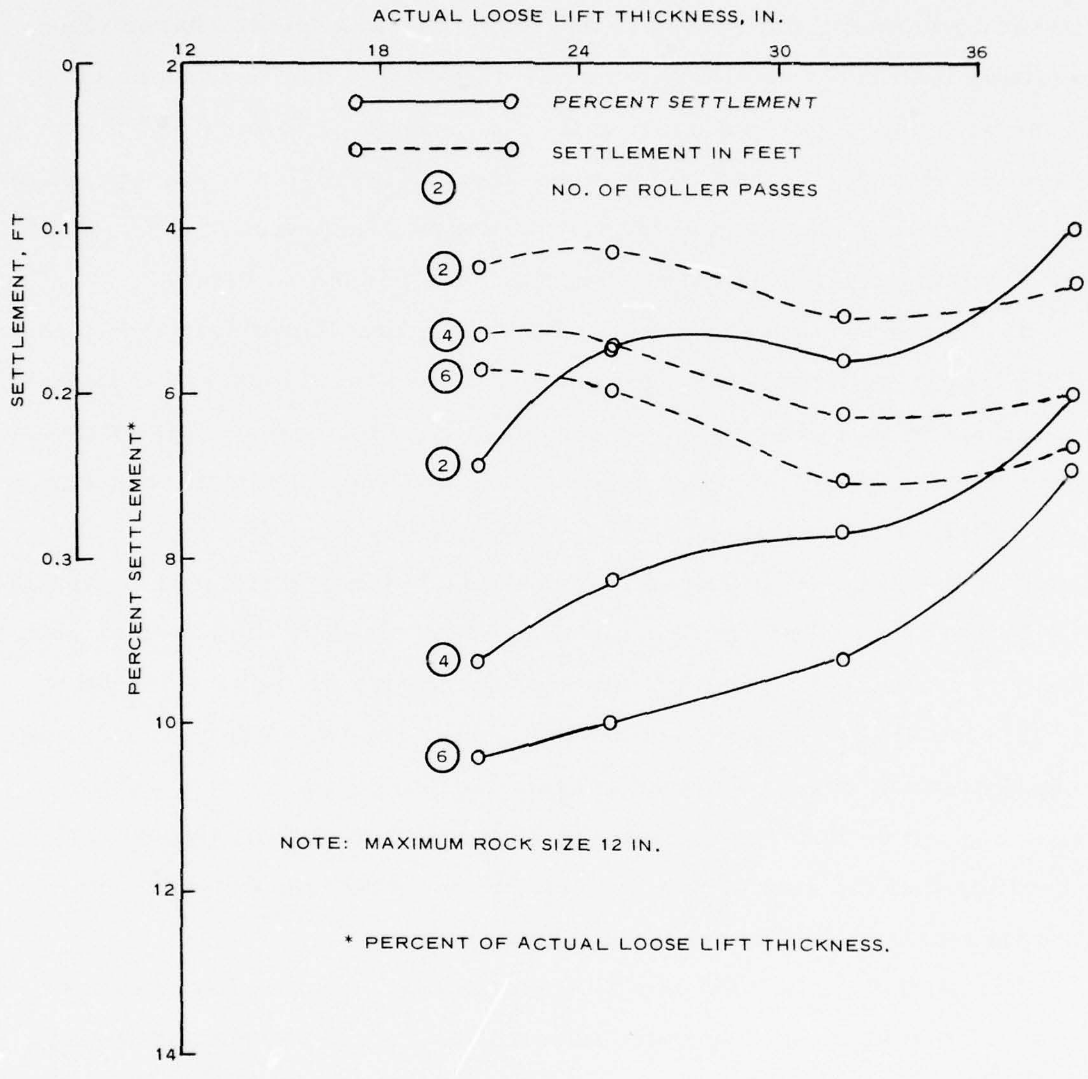


Fig. 30. Settlement in percent and feet vs actual loose lift thickness test fill series No. 3, Cougar test fill

reasonable upper bound for the loose lift thickness.

57. Both sets of curves do indicate that the majority of the compaction is attained by four passes; i.e., over 85 percent of the compaction obtained by six passes of the roller is attained in four passes.

Discussion

58. These tests clearly indicate the superiority of the 10-ton vibratory roller over all others tested, including the 5-ton vibratory roller, for this type rock. Also indicated is the fact that maximum size rock has an influence for a given rock and roller. However, more testing would be required to establish a reasonably definite relationship between all variables involved.

59. As a result of these test fills, specifications for the dam construction were modified to specify four passes by the 10-ton roller and to increase the loose lift thickness from 18 to 24 in.

New Melones Dam, Sacramento District

General

60. This test fill program was planned and construction in conjunction with the design of New Melones Dam, Stanislaus River, California. The program, which included four different rock fills, was conducted under contract with supervision by Construction Division personnel and technical assistance by the Engineering Division, Sacramento District. Construction of test fills 1 and 2 took place between 8 August 1968 and 16 August 1968; while test fills 3 and 4 were constructed during the period 10 October 1968 to 1 November 1968. The descriptive information and data contained herein were largely taken from Supplement No. 1 to Design Memorandum No. 23, "Embankment and Spillway Design," August 1970, prepared by the Sacramento District.

Rock type

61. A test quarry was utilized to obtain rock for the New Melones test fills. Rock encountered in the test quarry was gray to greenish gray, fine-grained, silicified metasandstone (a variant of the metavolcanics). Quarrying was done on three 30-ft benches with highly weathered to moderately weathered rock taken from the upper two benches and fresh unweathered rock taken from the lowest level. Blasting in the highly weathered zone produced considerable fines. The fresh rock was generally hard and durable and broke into irregular blocks and fragments with sharp edges. As is discussed in later paragraphs, the materials were separated into minus 6-in. and 6- to 18-in. sizes for placement in the fills. No provisions were made to separate the oversize fraction (that greater than 18 in.) except through selective removal during loading, nor were there any provisions to screen the fines from the minus 6-in. fraction.

Description of test fills

62. The test fills were constructed on a leveled area approximately 900 ft northeast of the quarry site. A typical plan and section of the side-by-side test fill layout are shown in fig. 31. The test lanes of each fill measured 30 ft in width and 55 ft in length. Ramps having a slope of 1V on 6H were provided at each end of the fills for access. Turnaround areas for the equipment were provided beyond the test fill limits. Rock was placed in each zone according to the schedule given in the tabulation below:

Test Fill No.	Rock Size	Nominal Loose Lift Thickness, in.	No. of Lifts	Construction Material
1	6 to 18 in.	18	4	Weathered rock
2	Minus 6 in.	12	6	Weathered rock
3	Minus 6 in.	12	6	Fresh rock
4	6 to 18 in.	18	4	Fresh rock

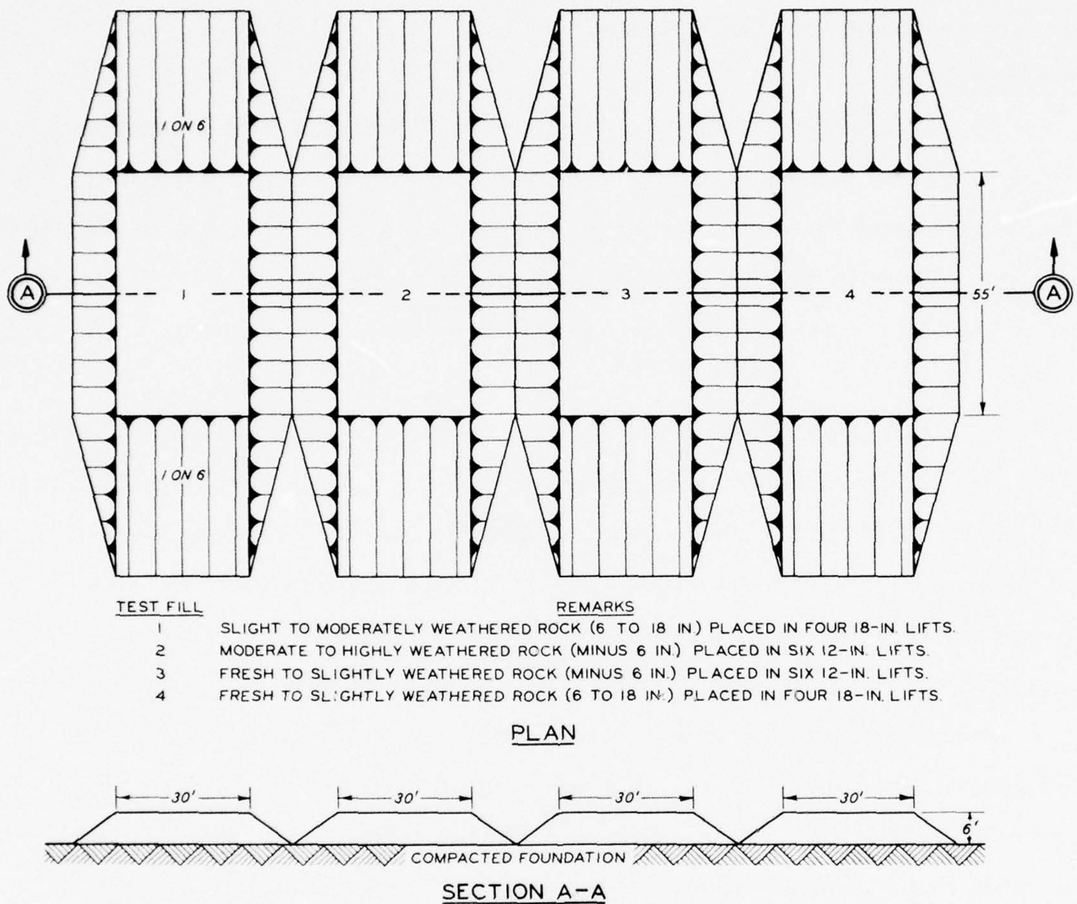


Fig. 31. Plan and profile of New Melones test fill

Thus plus and minus 6-in. fractions of both the weathered and fresh rock were tested. Separation was accomplished by the use of a vibrating grizzly with a 6-in. bar spacing.

Construction

63. Preceding fill placement, residual soil and highly weathered rock were stripped to provide a reasonably sound rock foundation. The foundation was then compacted by eight passes of a Bros Model VP-20D 10-ton vibratory roller. Settlement plates were not installed at foundation level since it was assumed that no significant foundation settlement would occur.

64. Rock for the fills was hauled from the quarry in trucks, dumped approximately in place, and spread to the approximate desired thickness by a D-8 bulldozer. The pattern of equipment traffic was arranged to minimize traffic over the test area. An attempt was made whenever possible to follow ordinary operating procedures used in placing and compacting fill in a dam. By dumping the rock approximately in place, the spreading distance was kept to a minimum, which lessened the chances of segregation. Only a very small percentage of oversize rock was delivered to the fills. Rocks sufficiently large to interfere with compaction (i.e., protruding well above the fill surface) were pushed to the outside, away from the test area.

65. After each lift was spread uniformly to the desired thickness, it was subjected to one complete coverage of the 10-ton vibratory roller with the vibratory unit off in order to smooth out the lift surface. This facilitated laying out a grid of elevation reference points and taking the initial level readings to establish the actual loose lift thickness. Compaction of the lift was then begun using the 10-ton vibratory roller vibrating at 1350 vpm and towed by a D-6 bulldozer at a speed of 1 to 1-1/2 mph. The roller was towed over the test lane in alternate

directions. The lift was subjected to a total of six passes by the roller. After rolling was finished, the lift surface was covered with a lime marker material to facilitate later identification when the fill was trenched for visual examination of the compacted material.

Tests and measurements

66. Procedures

a. Settlement measurements. Vertical settlement was used as a measure of relative compactive effort. The grid established on the surface of each lift to mark the points of settlement readings is shown in fig. 32. These points (24 in number) were spotted with spray paint on the surface of each lift after one pass of the roller with the vibratory unit off. Level readings were then taken to establish the actual loose lift thickness and subsequent readings were taken to determine the settlement after every two passes of the roller. To aid in obtaining an average elevation at each point, a steel plate with a raised button in the center (similar to the one used at Cougar Dam and shown in fig. 27) was placed on the surface over the point.

b. Mechanical analyses. Gradation tests were performed on representative samples taken before spreading and on samples of material excavated from the walls of the inspection trench cut through test fills 2 and 3. Each grading consisted of a 5- to 8-cu yd sample that was sorted and separately weighed into minus 4-, 8-, 12-, and 18-in. sizes. The separation was accomplished by dumping over screens made up of welded rebars into steel bins. Approximately 2-1/2 hours was required to grade each sample, which weighed about 30,000 lb.

c. Inspection trench. At the conclusion of construction, a trench was cut transversely through all four fills to observe the degree and uniformity of compaction, the amount of particle breakdown, and the lift-to-lift bonding. This trench, shown in fig. 33, was about 6-ft wide and was excavated by a front-end loader. The disturbed rock along each cut face was removed by hand to expose as undisturbed a face as possible. The marked surface of each lift was discernible in the vertical face and a good indication of rock sizes,

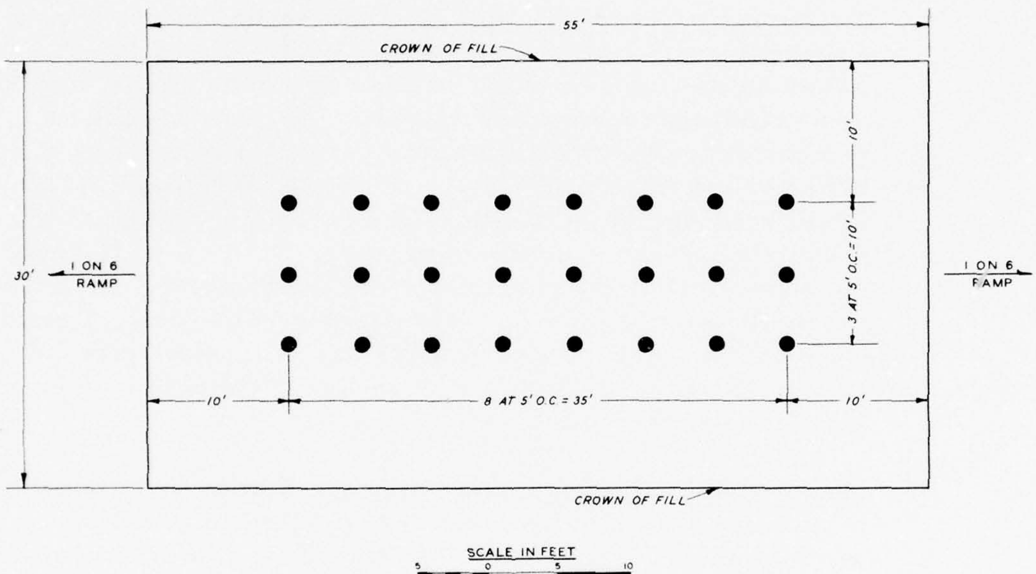


Fig. 32. Grid for settlement readings, New Melones test fill

View of transverse cut through rock test fill embankments. From front to back:

Test Fill No.	Description
1	Weathered rock (6- to 18-in.)
2	Weathered rock (Minus 6-in. transition)
3	Unweathered rock (Minus 6-in. transition)
4	Unweathered rock (6- to 18-in.)

Fig. 33. Inspection trench, New Melones test fill

density of the compacted lifts, and stratification between lifts was afforded by visual observation.

67. Results

a. Settlement measurements. Figures 34 through 37 show the percent settlement curves for each lift and the average percent settlement of all lifts in a particular fill. The settlement curves for each lift were obtained by averaging the level readings obtained at all points for a particular interval of rolling (i.e., after two passes, four passes, and six passes) and dividing by the actual loose lift thickness. The average settlement curves for each fill were obtained by averaging the settlement curves for each lift in that particular fill. The four average curves are plotted together in fig. 38 in order to better compare the settlement characteristics of the different fills. Because of unusual conditions associated with the construction of lift three (test fill 1) and lifts five and six (test fill 3), these data are not included in the calculated average curves. In lift three of test fill 1, numerous voids and segregation of the material within the test section occurred during dumping and spreading. This condition produced abnormally high settlements during rolling. Processed (grizzled) fresh rock used for the construction of lifts five and six of test fill 3 was saturated by rain prior to fill placement. This added moisture (retained by the finer fraction) appears to have affected roller compaction to the extent that a 1 to 2 percent increase in measured settlement resulted.

b. Mechanical analyses. Results of mechanical analyses performed on samples from test fill 2 (minus 6-in. weathered rock) and test fill 3 (minus 6-in. fresh rock) are shown in figs. 39 and 40, respectively. No mechanical analyses were performed on the 6- to 18-in. rock of test fills 1 and 4. The plots in figs. 39 and 40 show gradations of material before spreading and after rolling in order to indicate the degradation produced by fill operations. The results of the analyses are discussed below:

(1) The before-spreading curve for the weathered material of test fill 2 shown in fig. 39 is based on one sample. The after-compaction gradation curve is the average of four samples taken from the sidewall of the inspection

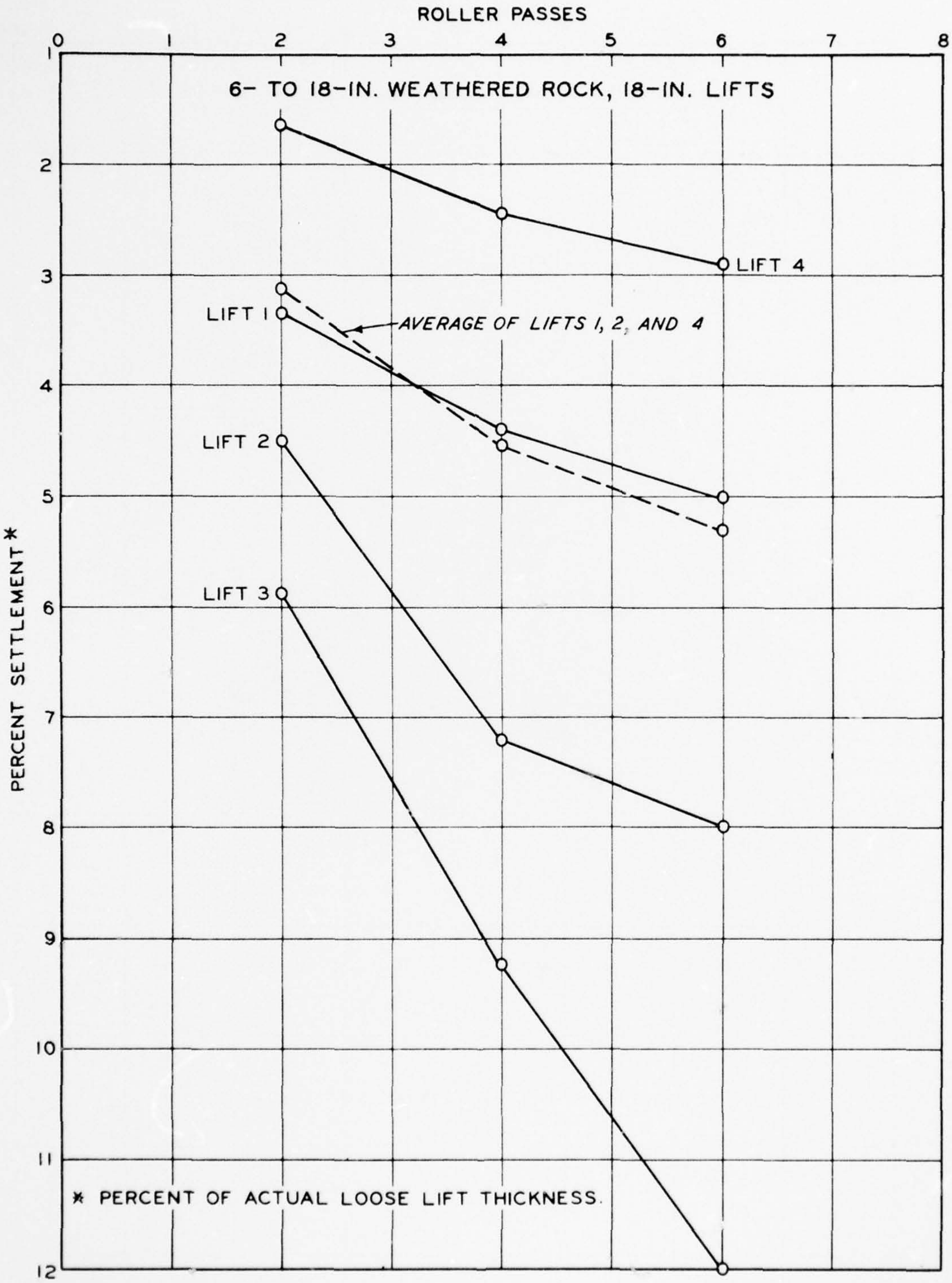


Fig. 34. Percent settlement vs roller passes, New Melones test fill 1

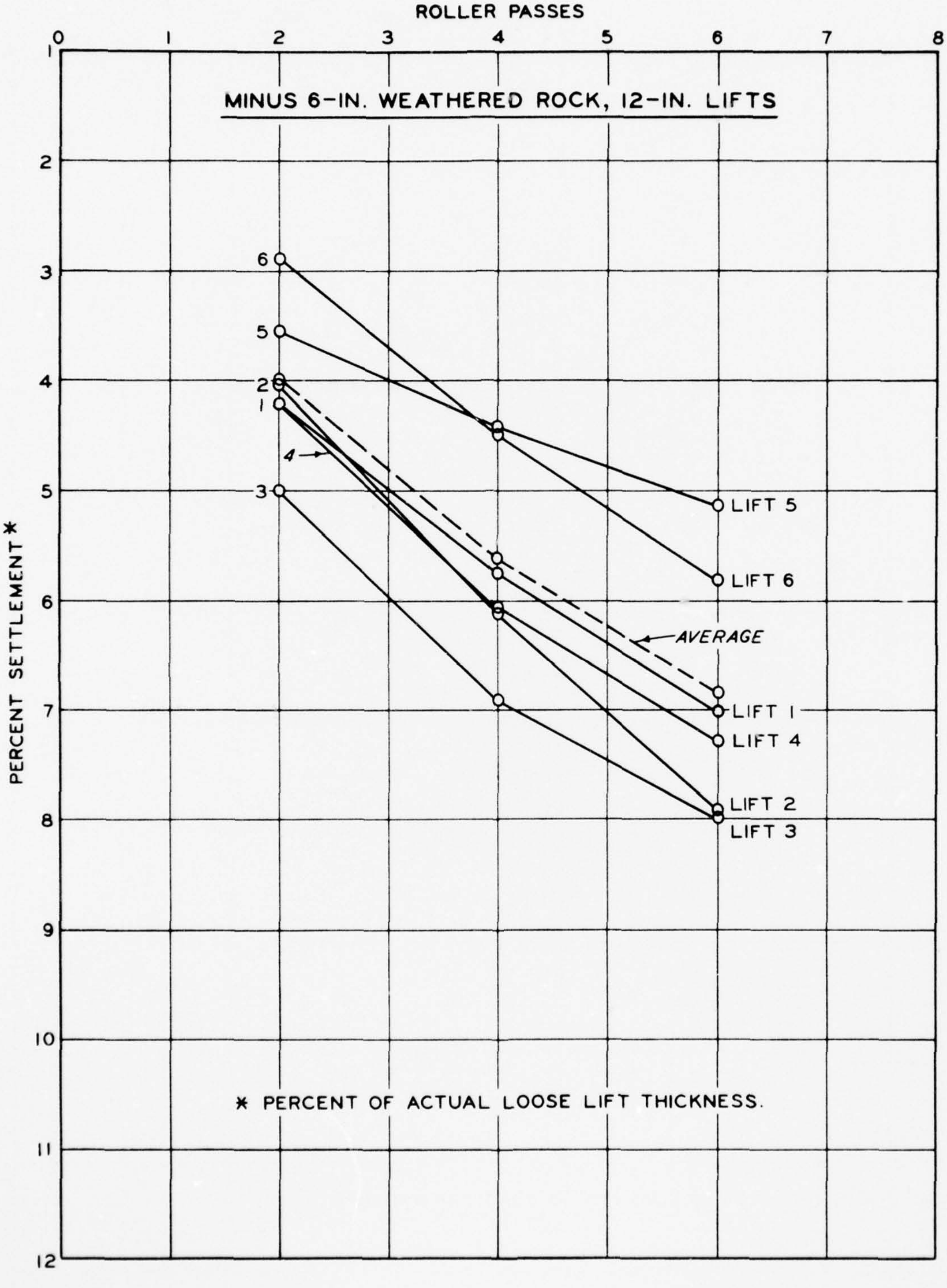


Fig. 35. Percent settlement vs roller passes, New Melones test fill 2

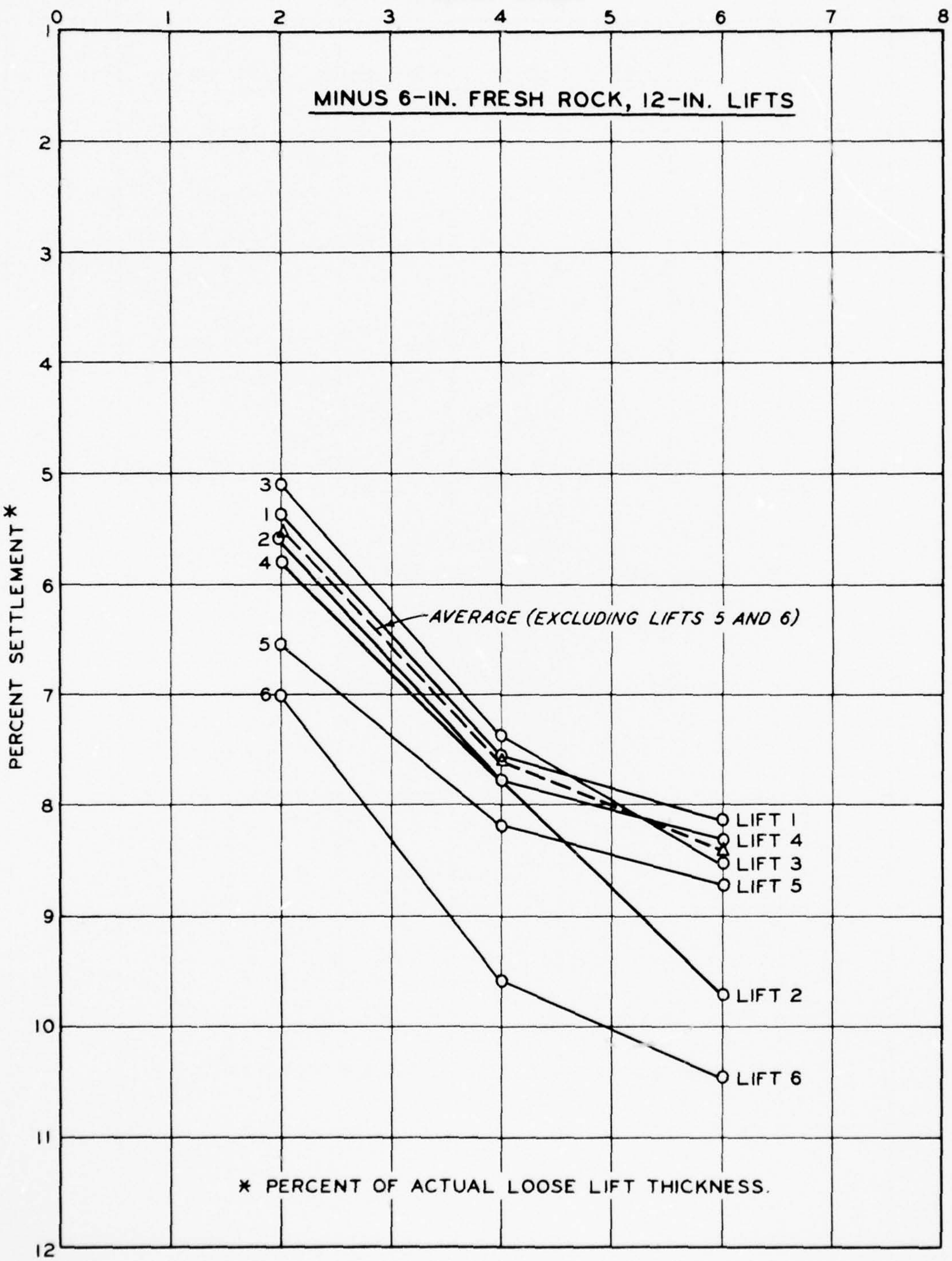


Fig. 36. Percent settlement vs roller passes, New Melones test fill 3

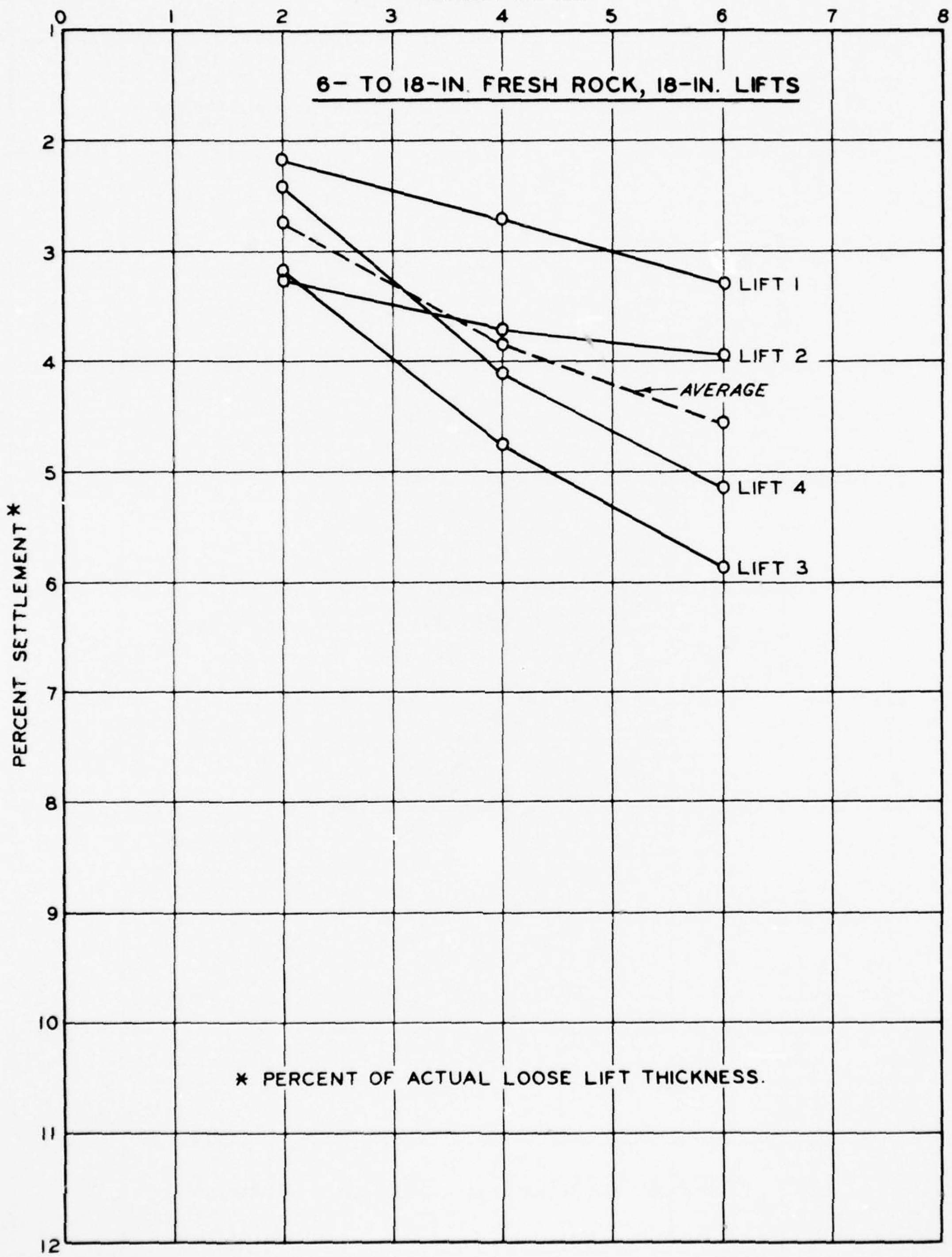


Fig. 37. Percent settlement vs roller passes, New Melones test fill 4

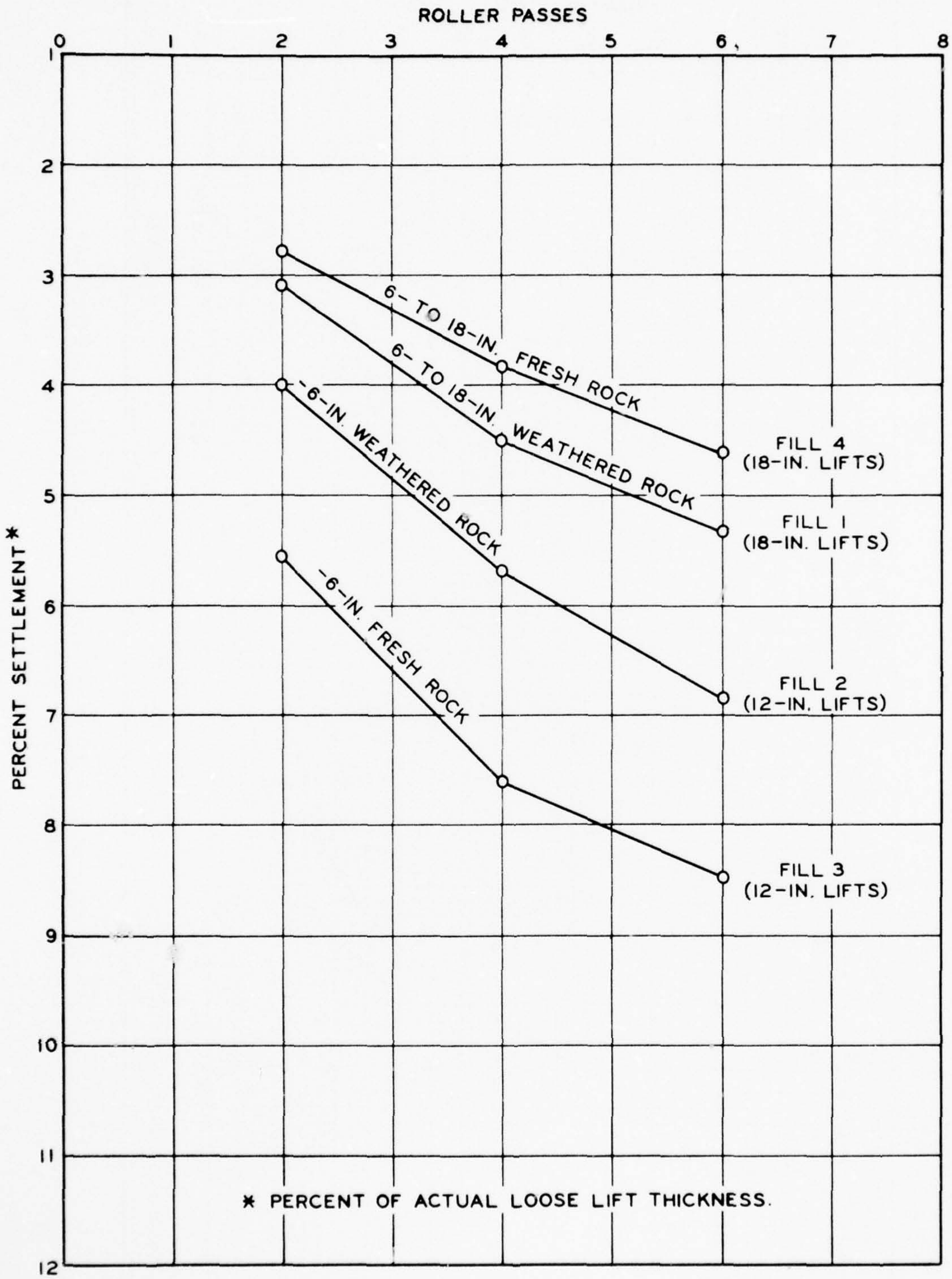


Fig. 38. Average percent settlement vs roller passes, New Melones test fills 1, 2, 3, and 4

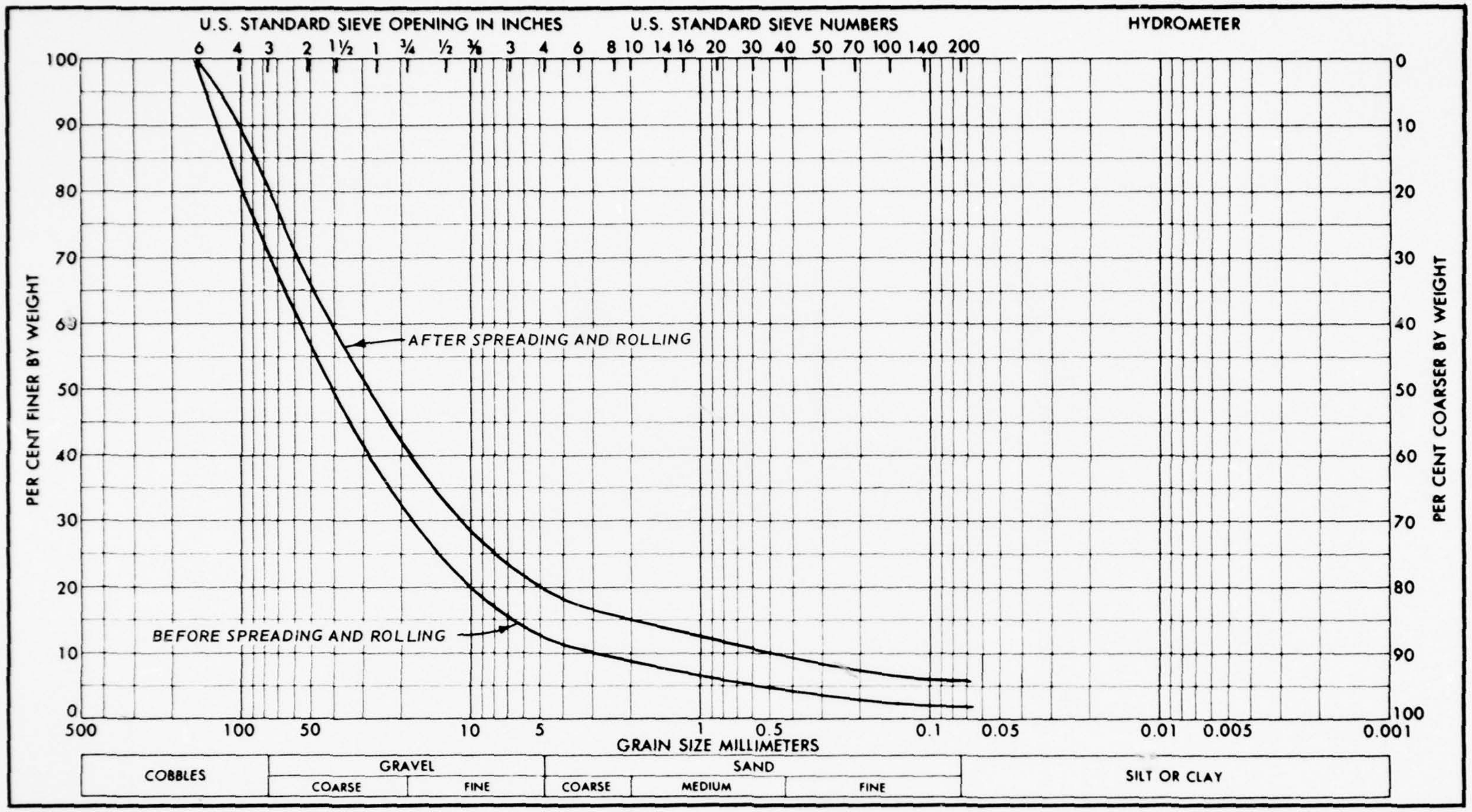


Fig. 39. Before spreading and after compaction gradation curves, test fill 2, New Melones test fill

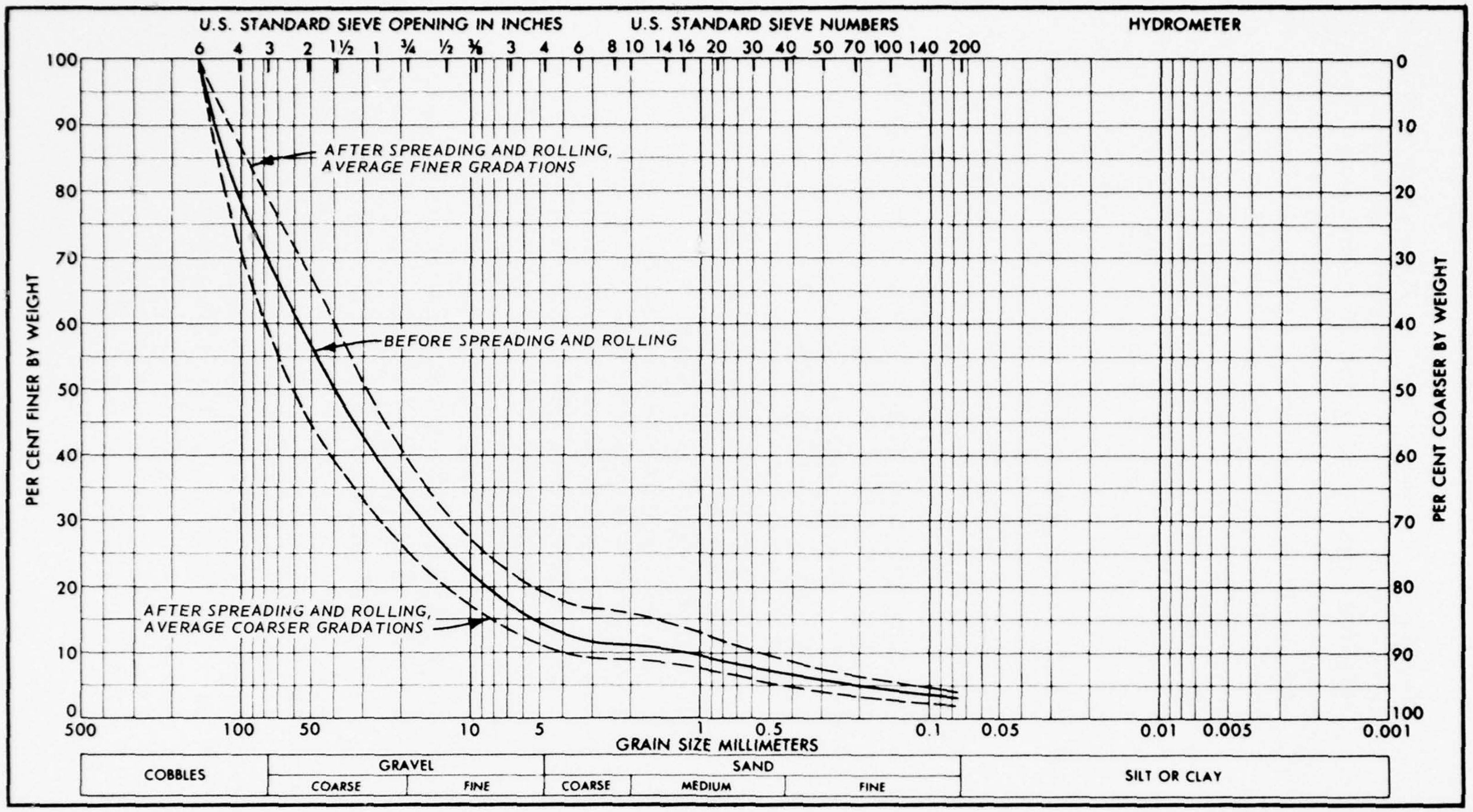


Fig. 40. Before spreading and after compaction gradation curves, test fill 3, New Melones test fill

trench. It is seen from these two curves that fill operations caused 5 to 10 percentage points increase in percent finer over most of the gradation range, with about 4 percentage points increase in fines (minus No. 200 sieve). It was noted by field personnel that the compaction of this material reduced surface fragments of each lift to a fine-grained soil. This would suggest that the degradation was primarily concentrated in the upper portion of the lifts. This is discussed later in connection with visual observations of the inspection trench sidewalls.

(2) The before-spreading gradation curve for the fresh rock of test fill 3 shown in fig. 40 is also based on only one sample. The two after-compaction gradations are average curves for a total of 8 samples taken from the cut face of the inspection trench. Four of those 8 samples had gradations that were finer than the before-compaction sample and four had gradations that were coarser than the given initial gradation. Since some stratification was observed within lifts for this fill, it is likely that the coarser gradation for the after-compaction case was the result of segregated material. If the average finer gradation curve is assumed to represent results of degradation, it is seen that a 5 to 10 percentage point increase in percent finer than any given sieve size occurred over most of the gradation range, but only about a one percentage point increase occurred in the minus No. 200 fraction. These gradation results point up difficulties inherent in obtaining representative samples from compacted rock fill.

c. Inspection trench. Visual examination of the trench walls of the fills resulted in the following observations:

(1) A considerable amount of small broken rock existed in the voids of test fills 1 and 4 (6- to 18-in. rock sizes). Inasmuch as the rock had been grizzled before being placed, the material finer than 6 in. must have been produced by spreading and rolling. Visual observation of the action of the roller indicated that protruding rocks and corners of larger rocks were readily broken. Occasionally, medium-sized rock was shattered into gravel sizes, and larger rocks protruding out of the fill surface

were punched down into the smaller surrounding rock. Observation of the compacted rock fill in the inspection trench revealed a marked uniformity of appearance throughout the entire depth of the fills. There were no planes of demarkation at the lift boundaries, and the rock was well interlocked. Fines appeared to be well distributed in the voids. Photographs of the sections through test fills 1 and 4 are shown in figs. 41 and 42.

(2) An examination of test fills 2 and 3 (minus 6-in. rock) revealed some tendency toward stratification of the rock fragments. Dumping and spreading operations were probably the major cause of the stratified appearance; although it was noticed that during compaction of test fill 2 (weathered material), the surface material was reduced to a fine-grained material similar to a soil. As the rock was dumped and spread by dozer, the coarse rock rolled ahead and to the sides of the pile, resting on top of the previously compacted surface of the lift below and forming a thin layer of segregated coarse material at the lift boundary. Interlocking between lifts was generally poor, due to the coarse material overlying the extremely smooth surface of the underlying lift. Photographs of the trench through test fills 2 and 3 are shown in figs. 43 and 44.

Discussion

68. Very little comparative data were available from these test fill operations as the objective was not to compare equipment types or loose lift thickness. The primary purpose of this test series was to observe the compaction characteristics of the two types of rock (weathered and fresh) over two gradation ranges.

69. The settlement data collected did indicate the effect of number of roller passes. While the average settlement curves in figs. 34 and 36 showed breaks in slope after four passes of the roller, signifying less settlement between four and six passes than that obtained between two and four, six passes (the maximum used) were insufficient to show limitations in benefits from increasing the number of passes. The

a. Section of rock test fill 1
(6- to 18-in. weathered rock)

b. Closeup view of rock test fill 1
(6- to 18-in. weathered rock)

Fig. 41. Inspection trench, test fill 1, New Melones test fill

a. Section of rock test fill 4
(6- to 18-in. unweathered rock)

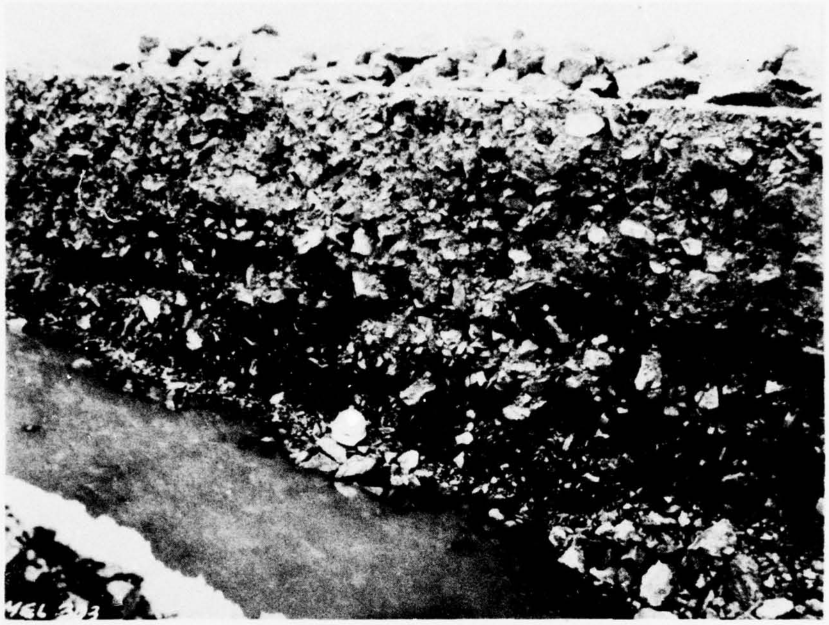
b. Closeup view of rock test fill 4
(6- to 18-in. unweathered rock)

Fig. 42. Inspection trench, test fill 4, New Melones test fill

a. Section of rock test fill 2
(minus 6-in. weathered rock transition)

b. Closeup view of rock test fill 2
(minus 6-in. weathered rock transition)

Fig. 43. Inspection trench, test fill 2, New Melones test fill



a. Section of rock test fill 3
(minus 6-in. unweathered rock transition)

b. Closeup view of rock test fill 3
(minus 6-in. unweathered rock transition)

Fig. 44. Inspection trench, test fill 3, New Melones test fill

minus 6-in. weathered rock of test fill 2, while indicating a slight reduction in settlement with increasing number of passes, did not show as pronounced a break as did the other fill materials. The before-spreading and after-rolling gradation curves complement the visual examination of the compacted material by means of an inspection trench. The weathered rock suffered the larger degradation as would be expected. The distribution of fines with depth within each lift could not be determined from the gradation curves as the gradation samples taken were representative of the total lift thickness. However, visual observation of the inspection trench sidewalls indicated a better distribution of fines throughout the entire lift for the plus 6-in. material. The minus 6-in. material appeared to be stratified, with the majority of fines found in the upper part of the lift. This is similar to results obtained at the Laurel test fill.

70. Based on the results of this test fill, the following requirements for construction of the dam were established:

a. Rock for the compacted rock zone should be retained on 6-in. horizontal splitter bars (plus 6 in.) and compacted in 18-in. loose lifts by four passes of a 10-ton vibratory roller.

b. Rock for the transition zone should be limited to 4-in. maximum size (i.e., material passing 4-in. horizontal splitter bars) and placed in 12-in. loose lifts, with each lift compacted by four passes of the vibratory roller.

Gillham Dam, Tulsa District

General

71. The purpose of the test fill program was to develop engineering data to be used in preparing plans and specifications for construction of Gillham Dam, Cossatot River, Arkansas. The primary objectives were as follows:

a. Size and gradation of rock-fill material obtainable from the

AD-A035 981 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MISS F/G 13/2
TEST FILLS FOR ROCK-FILL DAMS.(U)
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test quarry by varying blast-hole patterns and dimensions and by varying quantity and types of explosives.

b. Compaction of weathered and unweathered rock-fill materials by two different types of rollers (vibratory and pneumatic) using lifts of varying thickness.

c. Breakage and/or degradation of the rock-fill material due to rolling.

The test fill program consisted of one large rock-fill section having seven test panels or lanes (Nos. 1, 2, 3, 3A, 4, 5, and 6). Construction began on 19 June 1964 and was completed on 7 August 1964. Supervision for the project was furnished by the Construction Division, with technical assistance provided by the Engineering Division, Tulsa District. The information contained herein was largely taken from the Test Embankment Report, Gillham Dam, August 1964, and supplementary memoranda furnished by the Tulsa District.

Rock type

72. The Gillham test fill consisted of two types of rock: weathered sandstone and fresh sandstone. The weathered sandstone, distinguishable by its iron-stain color, was hard, dense, quartzitic, fine-grained, and slightly fractured. The weathering consisted of leaching and iron staining concentrated along bedding planes and joints. Blast fractures across the bedding and joint patterns revealed fresh blue-gray sandstone that was very hard and dense in the center of individual blocks. After blasting, the weathered material had angular particle shapes with about 10 percent of the rock degraded to sand sizes. The fresh or unweathered sandstone was very hard, brittle, and blue-gray in color. It appeared to be mildly metamorphosed with characteristics of quartzite. Scattered hairline fractures up to 1/8-in. wide were noted throughout this material. All of these fractures were tight to well healed with secondary quartz mineralization. Maximum rock sizes used in the test fill are given in

table 7.

Description of test fill

73. The test fill was constructed on a leveled area approximately 1600 ft northwest of the test quarry site. Preceding fill placement, about 4 ft of badly weathered shale was removed to expose a firm shale suitable as a foundation. Settlement plates were not installed at the foundation level.

74. The test fill consisted of seven sections or panels, as shown in fig. 45. Each panel was 18 by 50 ft, with 24- to 27-ft-wide (at completion) transition zones between panels. A 1V on 10H slope, serving as a ramp for the construction and compaction equipment, was provided on either side of the embankment. Panel 3A was added on the end of the fill (see fig. 45) after construction was underway to supplement erratic data obtained from panel 3. Rock type, maximum rock size, loose lift thickness, number of lifts, compaction equipment, and number of passes for the seven test panels are given in table 7.

Construction

75. Descriptions of excavation, hauling, and compaction equipment are given in table 8. Rear-dump hauling units, loaded at the test quarry by power shovel, transported the rock directly to the test fill (no stock-piling was allowed) and dumped it at the leading edge of the advancing loose lift. Spreading to the desired loose lift thickness was then accomplished by a D-8 bulldozer. Oversize rock was controlled in the quarry by selective loading. Any oversize material reaching the test embankment was wasted during spreading. The shale content of the material was kept to a minimum by selective removal where possible. However, much interbedding of shale in the sandstone was encountered, and selective removal was not always feasible. Material for the last three lifts of panel 5 and all of panel 6 was passed through a screen to

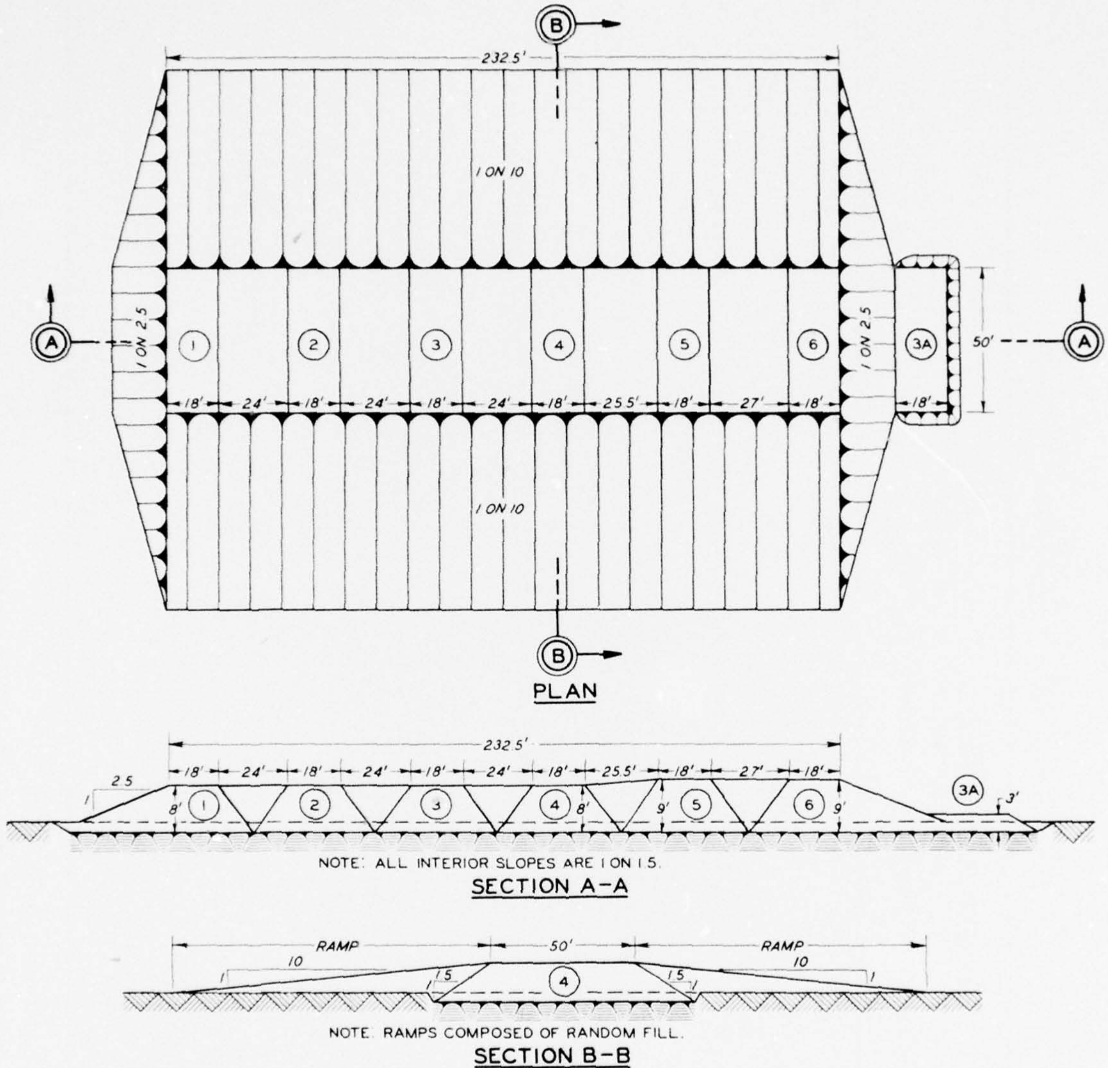


Fig. 45. Gillham test fill layout

Table 7

Construction Details, Gillham Test Fill

Panel	Material	Maximum Rock Size Weight lb	Maximum Rock Size Approximate Spherical Diameter* in.	Lift Thickness in.	No. of Lifts	Roller	No. of Passes per Lift
1	Weathered sandstone	80	11-1/2	12	8	Vibratory	8
2	Weathered sandstone	250	17	24	4	Vibratory	8
3	Weathered sandstone	80	11-1/2	12	8	Pneumatic	8
3A	Weathered sandstone	80	11-1/2	12	3	Pneumatic	8
4	Weathered sandstone	250	17	24	4	Pneumatic	8
5	Fresh sandstone**	200	16	18	6	Vibratory	8
6	Fresh sandstone (plus 3-in. only)	300	18	36	3	Vibratory	8

* Based on a specific gravity of 2.61
** Last three lifts had all minus 3-in. material removed

Table 8

Construction Equipment, Gillham Test Fill

Item	Function	Description
Power shovel	Loading	Lima Model 604, 1-1/2-cu yd bucket capacity
Trucks (3)	Hauling	Euclid Model R-15, rear dump (2) Euclid Model 91-FD, rear dump (1)
Tractor	Spreading	Caterpillar D-8 with dozer blade
Tractor	Towing	Caterpillar D-7 with dozer blade
Compactors (2)	Compacting	Bros Model VP-20D, 10-ton, vibratory Ferguson 50-ton pneumatic
Separator	Rock separation	3-in. screen with a wobble-type feeder adjusted to 3-1/4-in. openings and equipped with a belt-loader

remove the minus 3-in. fraction. All other material was quarry-run.

76. After spreading, each lift was smoothed by one pass of the Bros Model VP-20D 10-ton vibratory roller with the vibratory unit off or by one pass of the Ferguson 50-ton pneumatic roller, depending upon the specified method of compaction for that lift. A 6-ft grid pattern was then laid out on the smoothed lift surface from reference points beyond the test zone limits and subsequently marked with spray paint for easy identification. Initial level readings were taken on these points to establish the loose lift thickness (by comparing the new readings with the final readings on the underlying foundation or compacted lift). Compaction was then begun using either the vibratory or pneumatic roller, both towed by a D-7 bulldozer at speeds of 1 to 1-1/2 mph. The roller was towed over the test panels in alternate directions. Each lift was subjected to a total of eight passes by the roller, with level readings to measure settlement taken after every two passes.

Tests and measurements

77. Procedures

a. Settlement measurements. Measurements of settlement were the primary means of assessing compactive effort. The grid marked on the surface of each lift to delineate the 24 points of settlement measurement is shown in fig. 46. Initial level readings were taken as discussed above. An average elevation at each point of the grid was obtained by placing the level rod in the center of a 12- by 12- by 1/2-in. steel plate (similar to that shown in fig. 27) positioned over the grid point.

b. Density tests. Field density tests were made at the conclusion of construction in the top lifts of panels 3, 3A, 4, 5, and 6. The tests were performed by excavating through the top lift of the panel from within a 6- by 6-ft wooden guide frame placed on the surface. Excavation of the hole was by hand labor, with each of the larger rocks being weighed individually and the smaller particles in groups. An extensive number of tape measurements were made of the finished

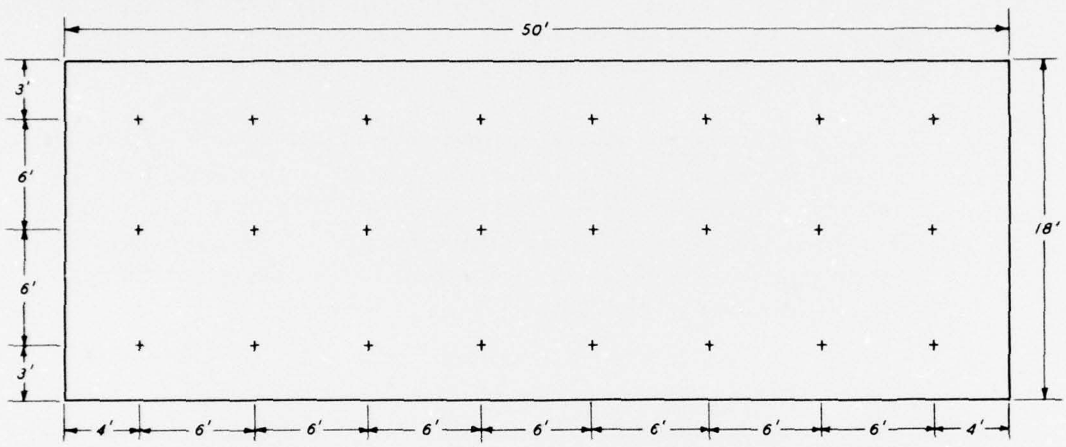


Fig. 46. Grid layout for level readings, Gillham test fill

pit and averaged in an effort to obtain accurate dimensions from which to compute the volume of the hole.

c. Mechanical analyses. Gradation tests were run on the material excavated from the density test pits in panels 3, 3A, 4, 5, and 6. These analyses established the after-compaction gradations for the top lifts of those five panels. These tests were run by first weighing the total sample, then grouping the rocks into weight ranges, and finally computing the percent of the total sample represented by each weight range. The percent smaller than 3 in. was determined by actually sieving the finer fraction over a 3-in. sieve.

d. Inspection trench. After completion of all panels of the test fill, an inspection trench was excavated with the D-8 bulldozer. The trench, which had a base width of 14 ft, exposed all lifts of all panels in the embankment. The inspection trench permitted visual assessment of the compaction characteristics of the fill.

78. Results

a. Settlement measurements. Figures 47 through 51 show the percent settlement of each lift of panels 1, 2, 3, 3A, and 4 which were constructed of quarry-run weathered rock. The average settlement of each of these panels is shown in the respective figures and in the plot of fig. 52. Figures 53 and 54 present the lift settlement data for panels 5 and 6, which were composed of plus 3-in. grizzled fresh rock except for lifts 1, 2, and 3 of panel 5, which contained quarry-run fresh rock. The average settlement curves for panels 5 and 6 are also shown in figs. 53 and 54 and plotted separately in fig. 55. The immediately noticeable aspect of the settlement data for the panels constructed of weathered rock (panels 1, 2, 3, 3A, and 4) is the erratic data for several of the lifts. The measurement data for panels compacted with the 50-ton pneumatic roller (3, 3A, and 4) were less consistent than those for panels rolled with the vibratory unit, which might be expected because of the much smoother surface left by the vibratory roller. Further examination reveals that the most variable results were for the 12-in. lifts, regardless of the compaction equipment used. The erratic nature of the settlement data from the 12-in. lifts of panels 1, 3, and 3A probably resulted from erroneous settlement readings or

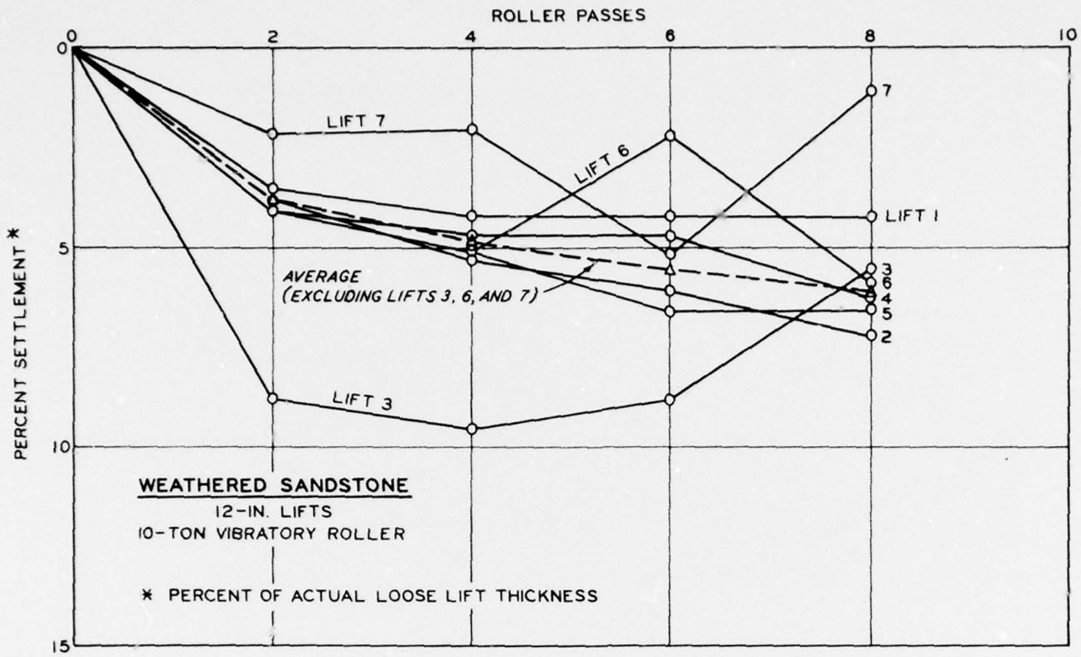


Fig. 47. Percent settlement vs roller passes, panel 1, Gillham test fill

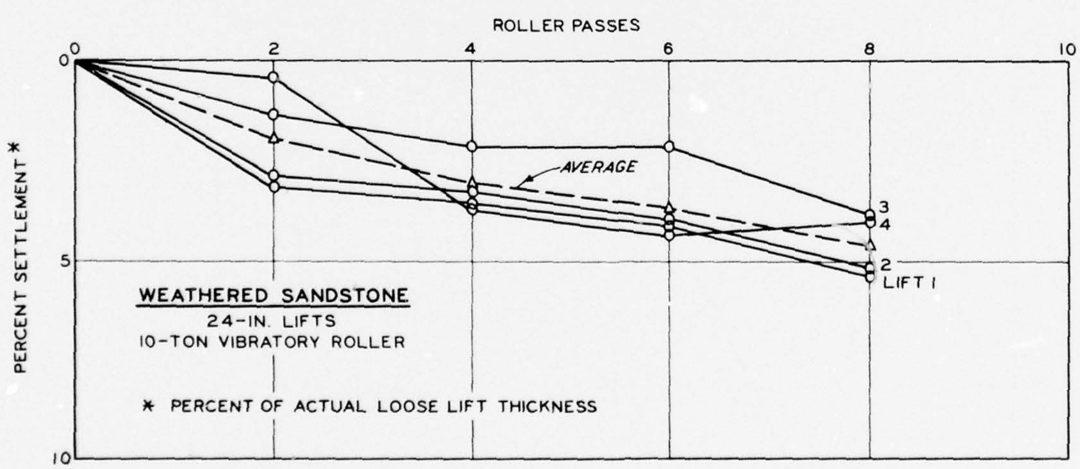


Fig. 48. Percent settlement vs roller passes, panel 2, Gillham test fill

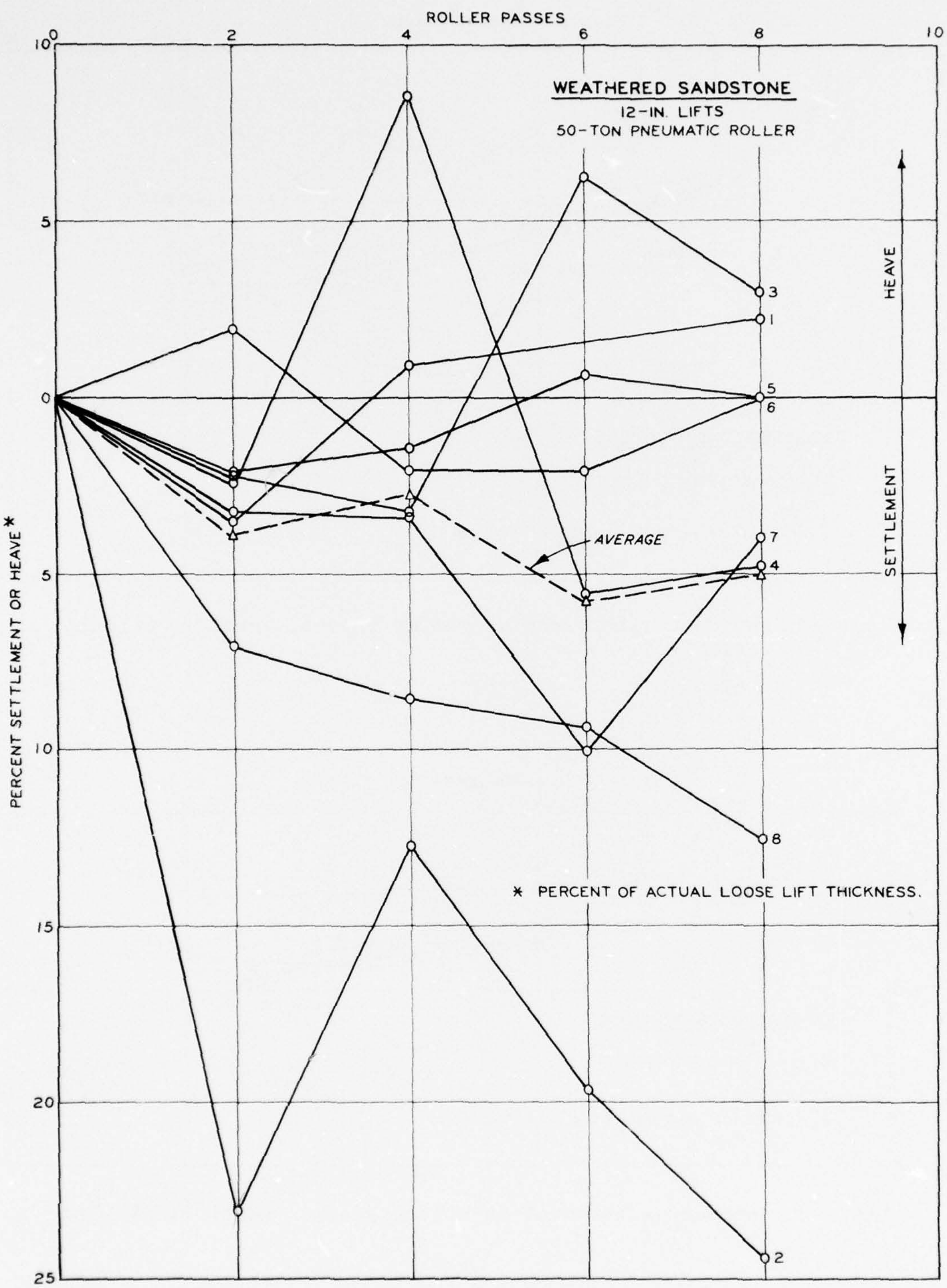


Fig. 49. Percent settlement vs roller passes, panel 3, Gillham test fill

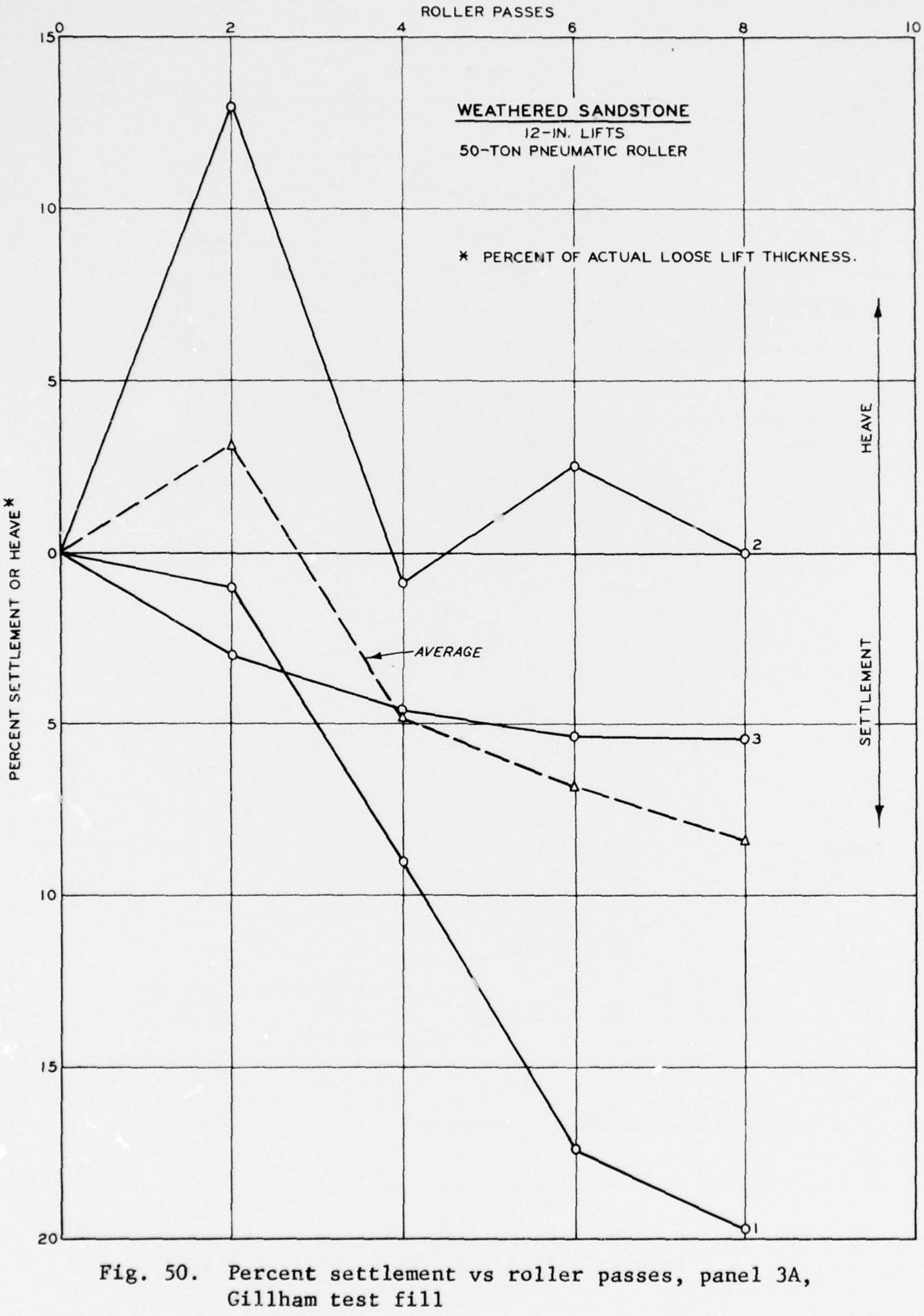


Fig. 50. Percent settlement vs roller passes, panel 3A, Gillham test fill

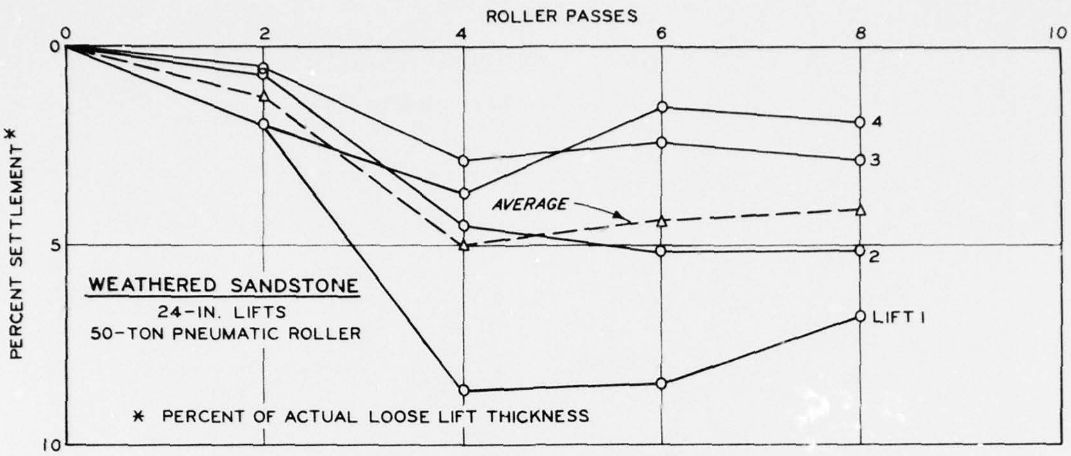


Fig. 51. Percent settlement vs roller passes, panel 4, Gillham test fill

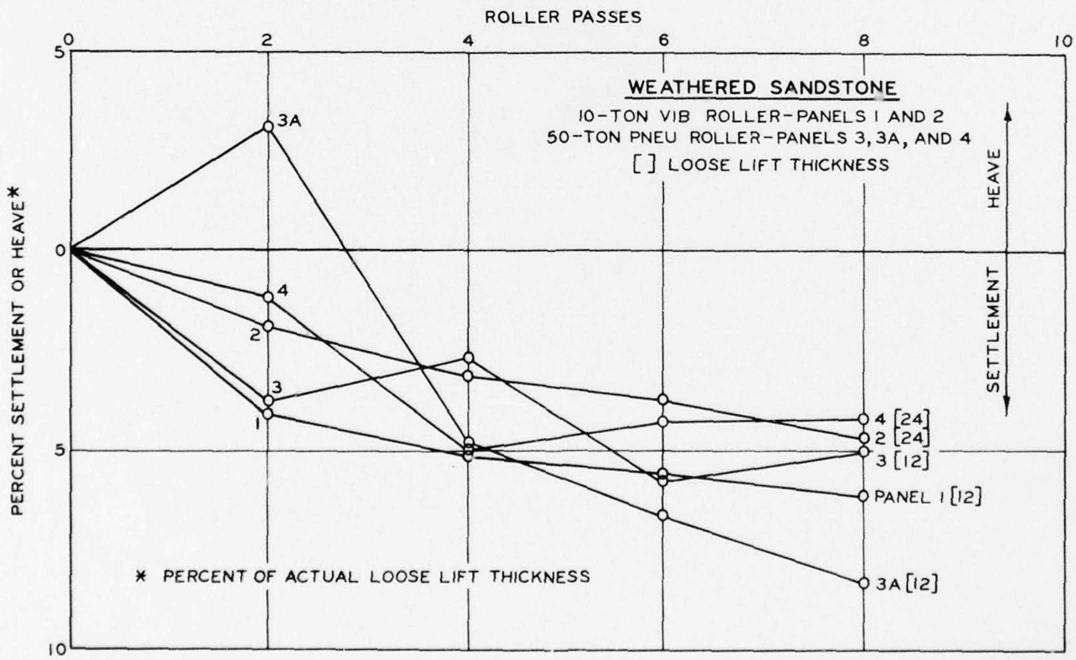


Fig. 52. Average percent settlement vs roller passes, panels 1, 2, 3, 3A, and 4, Gillham test fill

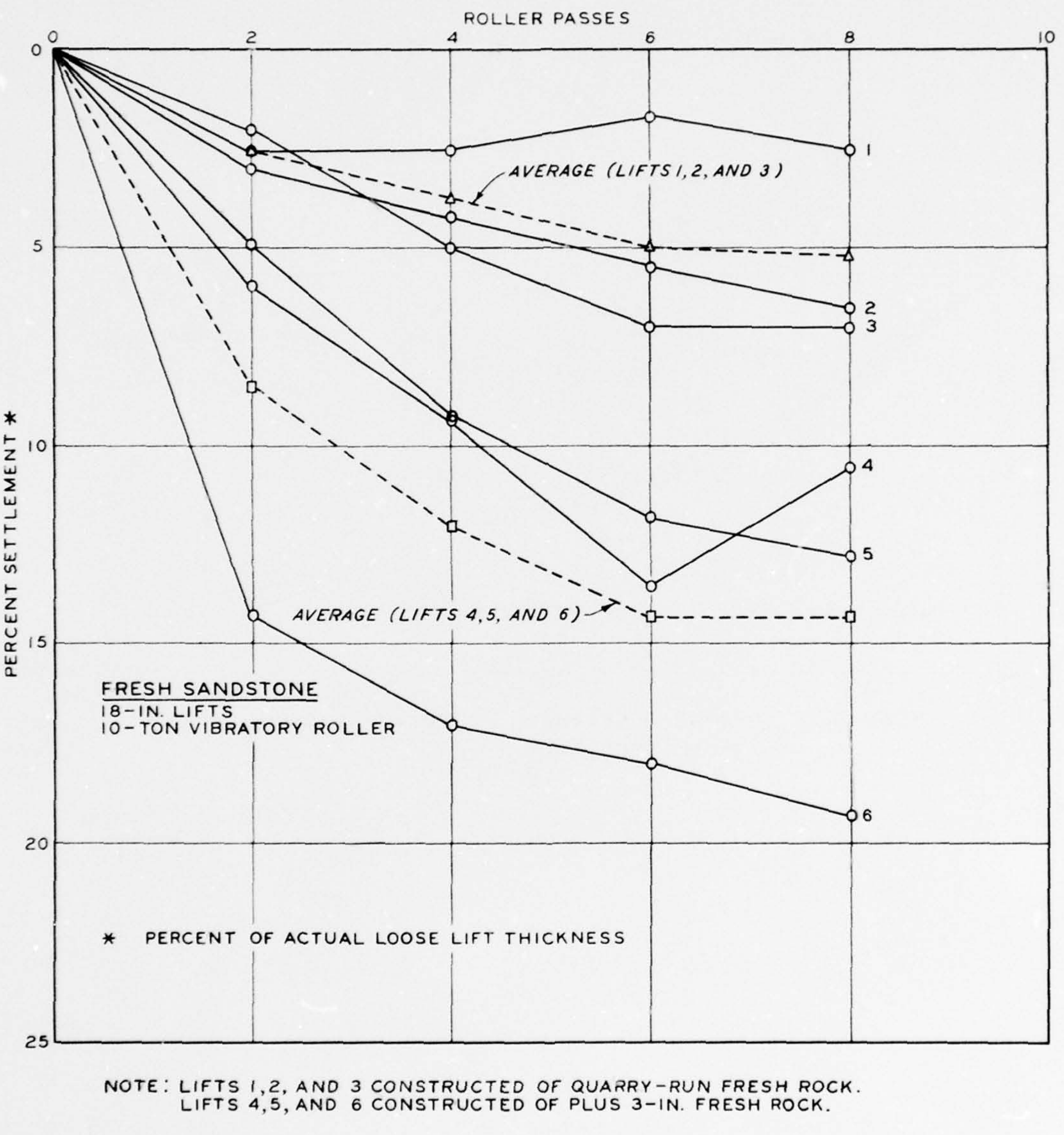


Fig. 53. Percent settlement vs roller passes, panel 5, Gillham test fill

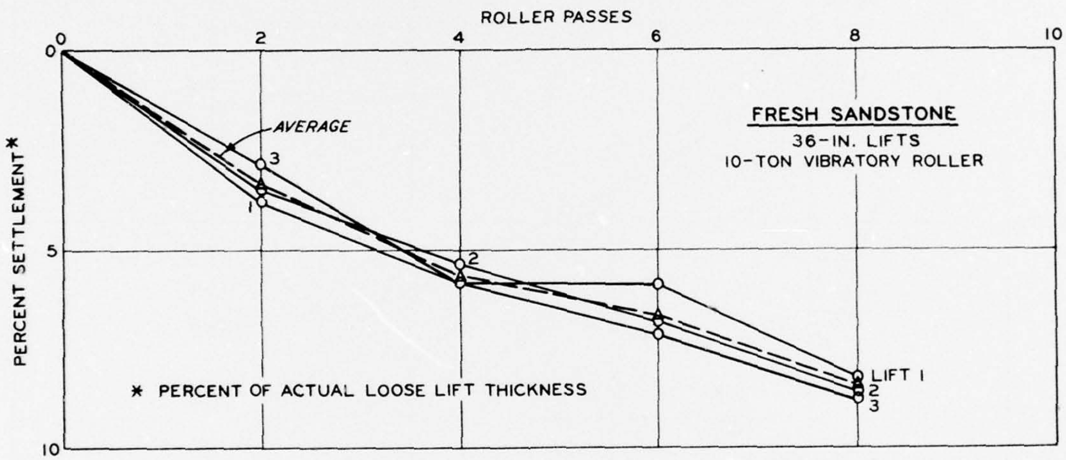


Fig. 54. Percent settlement vs roller passes, panel 6, Gillham test fill

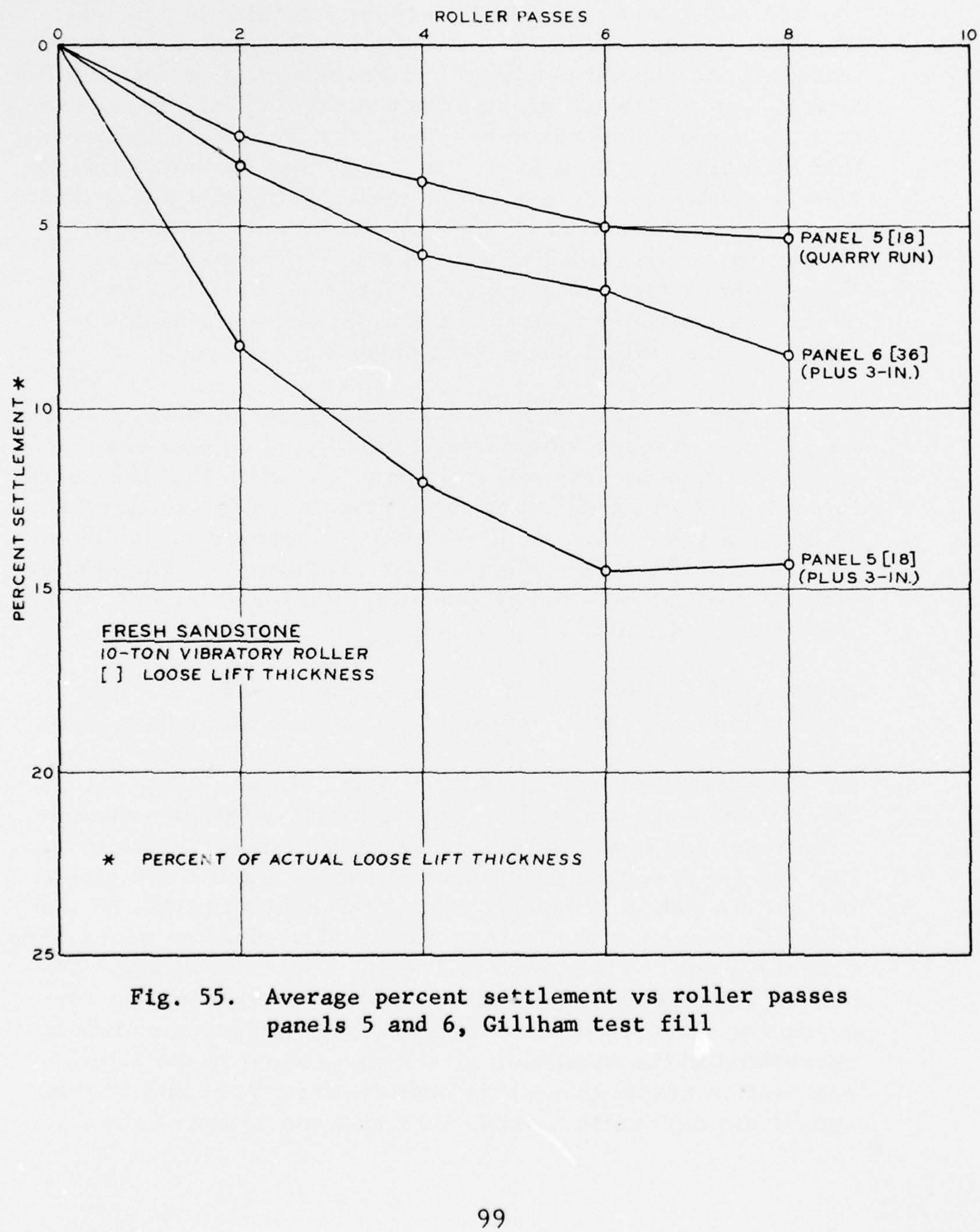


Fig. 55. Average percent settlement vs roller passes panels 5 and 6, Gillham test fill

the presence of oversize rock. The data (see fig. 52) indicate, however, that slightly better compaction was obtained with 12-in. lifts of weathered sandstone, and that four passes of either type roller produced most of the settlement achieved by eight passes for this lift thickness. The settlement plots for panels 5 and 6 (fresh sandstone), shown in figs. 53 and 54, are much less variable than those for the other panels. The data in figs. 53 and 55 indicate that the plus 3-in. material had superior compaction characteristics than did the quarry-run material. Comparison of figs. 53 and 54 indicates that the use of 18-in. lifts resulted in more efficient compaction than did the use of 36-in. lifts. Figure 54 shows that the rate of settlement for the 36-in. material had not decreased even after eight passes of the roller, and only 9 percent settlement was attained at that point. By comparison, fig. 53 shows that the same material in 18-in. lifts reached 14 percent settlement after six passes with a marked decrease in the rate of settlement beyond six passes.

b. Density tests. The results of density tests made in panels 3, 3A, 4, 5, and 6 are summarized in table 9. It was observed during the density tests taken in panels 5 and 6 that the rock-to-rock contact produced by the vibratory roller resulted in an unusually high degree of stability. Considerable pick work was required to loosen the rock for excavation. The vertical sidewalls of the pits had no tendency to slide or slough, but were tight and stable.

c. Mechanical analyses. After-compaction gradation tests were performed on the material taken from the density tests in panels 3, 3A, 4, 5, and 6. The after-compaction curves for material in panels 3, 3A, and 4 are shown in fig. 56. Since there were no before-compaction data for these panels, no assessment of particle breakage can be made. The gradation curves resulting from tests in panels 5 and 6 are given in figs. 57 and 58, respectively. Also shown in figs. 57 and 58 is the curve for the quarry-run fresh rock after processing over a wobbler grizzly with 3-1/4-in. screen openings. The curves representing the material before placement and compaction were derived from the processed quarry-run data by correcting to the maximum size rock present in the after-compaction gradations. It is seen from figs. 57 and 58 that significant degradation occurred under the action of the

Table 9

Results of Density Tests After Compaction

Panel No.	Lift Thickness in.	Material*	Roller	In Situ Density pcf	Porosity %
3	12	Weathered sandstone	Pneumatic	135	17
3A	12	Weathered sandstone	Pneumatic	128	21
4	24	Weathered sandstone	Pneumatic	137	16
5	18	Fresh sandstone	Vibratory	91	43
5	18	Fresh sandstone	Vibratory	83	48
6	36	Fresh sandstone	Vibratory	104	36

* All weathered sandstone was quarry-run; all fresh sandstone was plus 3-in. material.

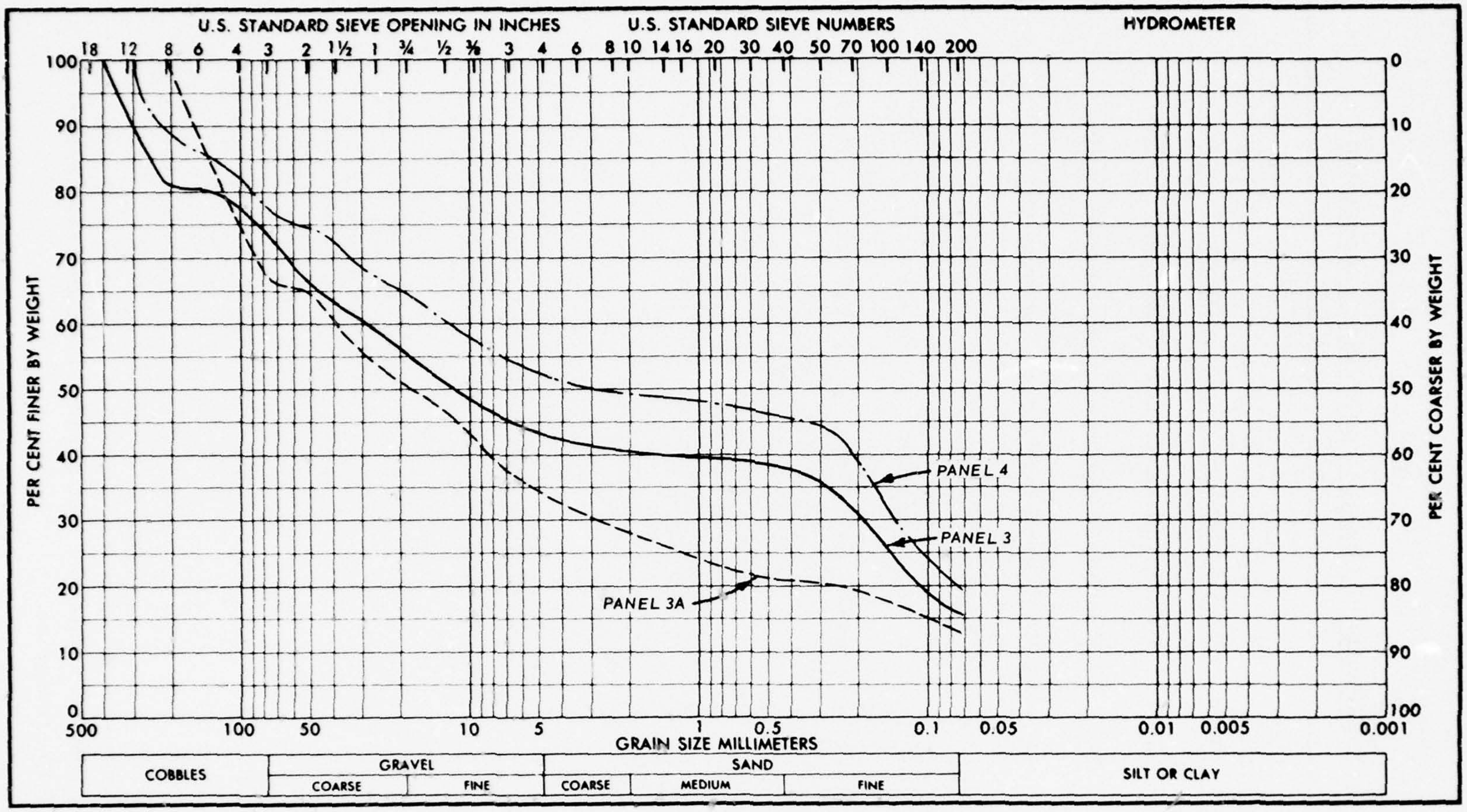


Fig. 56. After-compaction gradation curves, panels 3, 3A, and 4, Gillham test fill

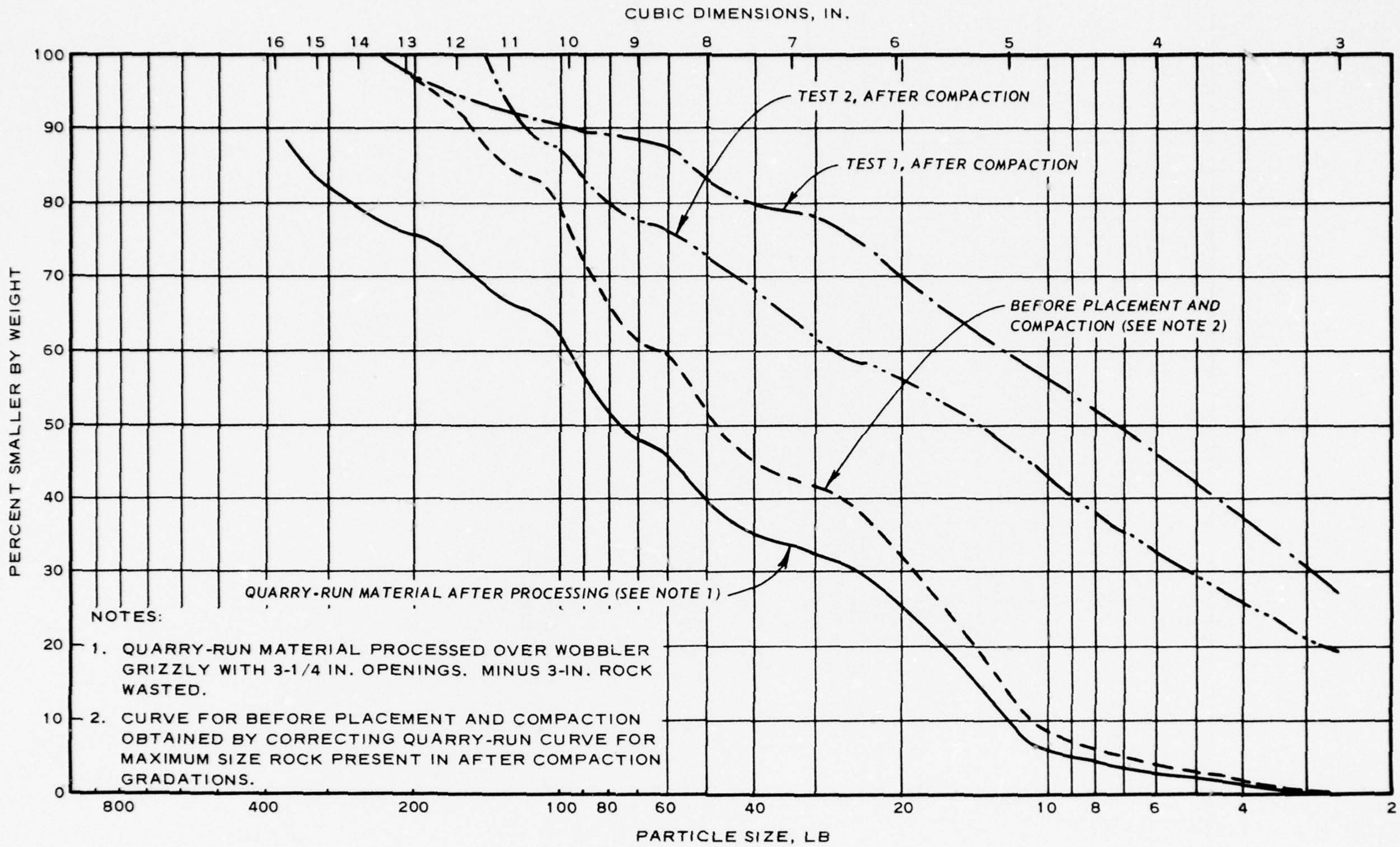


Fig. 57. Before- and after-compaction gradation curves, panel 5, Gillham test fill

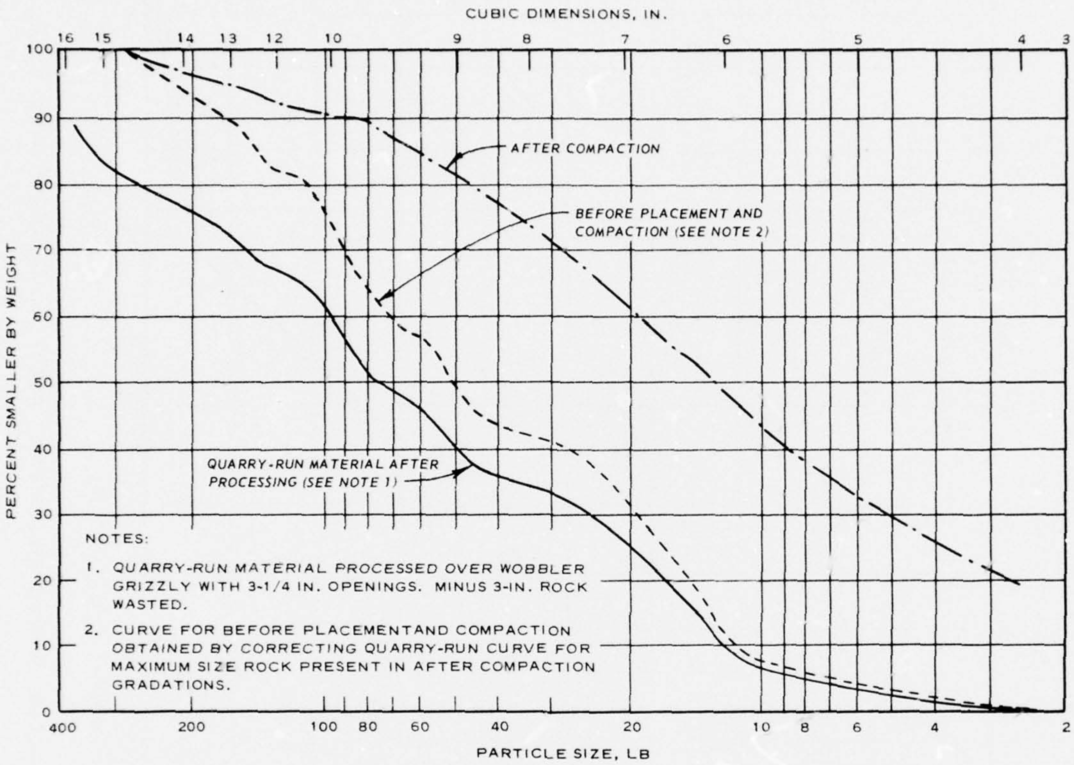


Fig. 58. Before- and after-compaction gradation curves, panel 6, Gillham test fill

vibratory roller. Field personnel noted that 85 percent or more of the larger size rock chunks were broken at least once by the vibratory compactor.

d. Inspection trench. Visual examination of the inspection trench confirmed the tightness and unusually high degree of stability of all compacted material. No mention was made of any segregated or stratified zones or that any evidence of poor bonding between lifts was found. Two views of the inspection trench are shown in the photographs in fig. 59.

Discussion

79. As mentioned earlier, the settlement data indicated more settlement was obtained using the plus 3-in. fraction rather than the entire (quarry run) portion of the fresh sandstone. This also occurred at the Laurel Dam test fill, and, although no mention was made by the district about stratification and excess surface breakage, it is probable that the cause was not only that there were more open voids in the plus 3-in. material to begin with, but that a dense layer of surface fines formed in the quarry-run material, which reduced the compaction obtained in the lower part of the lift as previously discussed in paragraph 27.

80. The results of the density tests on the fresh sandstone fill material (91, 83, and 104 pcf) seem low, especially in view of both the description given of the density pit sidewalls (see para 78b) and the settlement data. All things considered, one would tend to suspect the accuracy of the density tests. Large-volume density tests are themselves crude, even when the best possible procedures are used. In these tests, pit volume was determined by geometric measurements which is not desirable for a rock-fill material. Computations of volume made from rule or tape measurements usually fail to account for protruding rocks, cavities, and unsymmetrical pit shapes. It should be recognized, however, that it is possible to obtain a very stable,

a. View north, inspection trench excavation

b. View south, inspection trench excavation

Fig. 59. Inspection trench, Gillham test fill

free-draining, and otherwise desirable rock fill without necessarily achieving a correspondingly high density.

81. The erratic nature of some of the settlement data indicates the problems involved in taking level readings on a rock-fill surface. Often a premarked point for a reading will consist of rock that has rotated under the roller action rather than having been pushed down, and thus does not reflect the average elevation of the surrounding surface. This is often true in the case of compaction by rubber-tired rollers because they do not tend to crush the surface rock as does a vibratory roller. This problem might also be aggravated by the use of thinner lifts. Some inaccuracies could also be caused if significant settlement is still occurring in underlying lifts but is being measured as occurring in the lift under compaction. An alternative to this problem would be to plot the data as actual settlement for an entire panel (as long as all lifts in the panel were of the same thickness and material). However, if settlement by lift in terms of percent of initial lift thickness is desired, the preceding lift can be rolled until no significant additional settlement is observed.

Summary and Conclusions

82. Settlement data from the six test fill programs discussed in Part II are plotted in figs. 60 and 61. The purpose of these plots was to see if any trends developed for any of the many variables involved. However, an examination of figs. 60 and 61 reveals no such trends. There are simply too many variables involved, with rock type being one of the more influential.

83. Summaries of the variables evaluated and of the spreading, towing, and compaction equipment used in the six test fills are given in

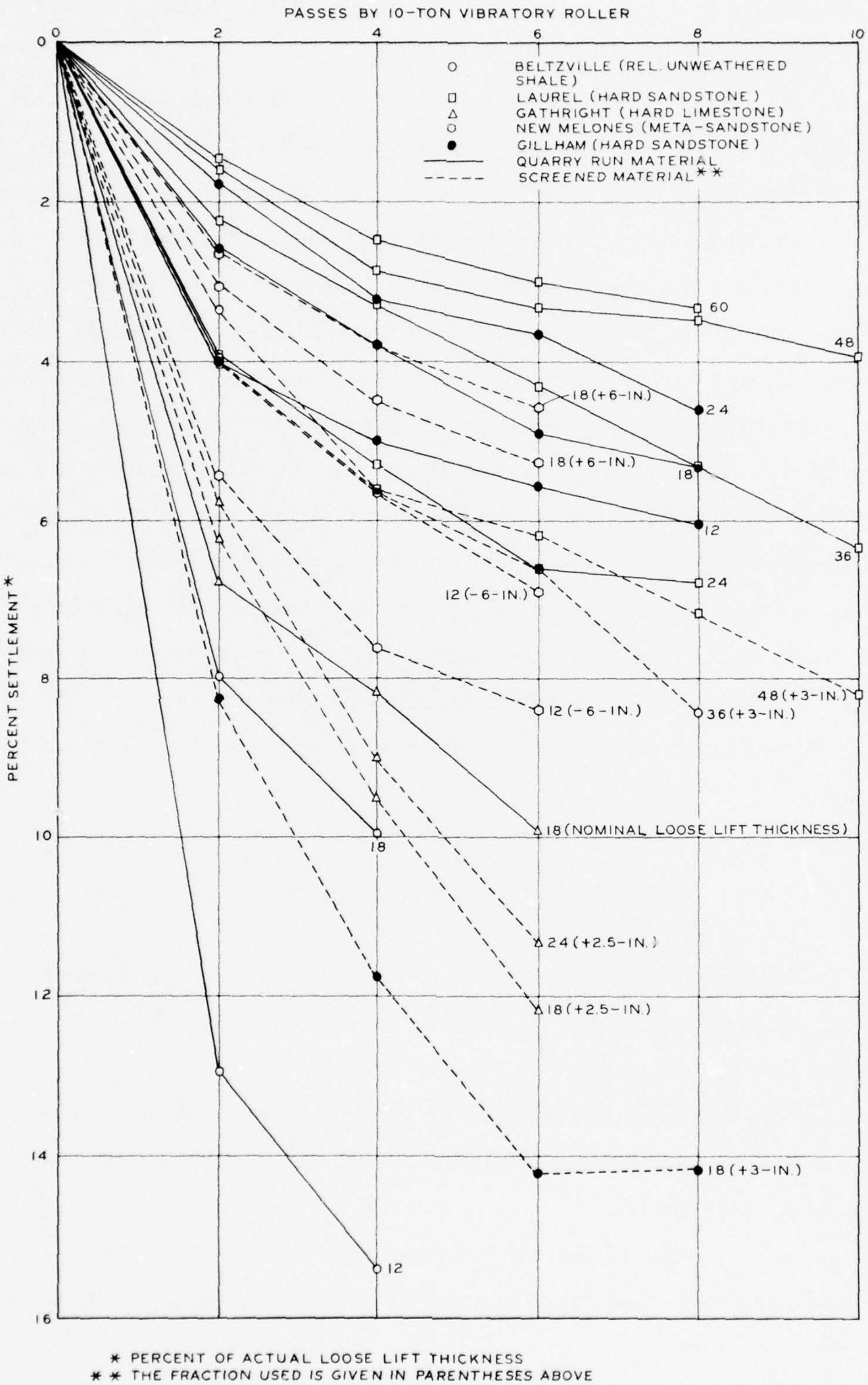
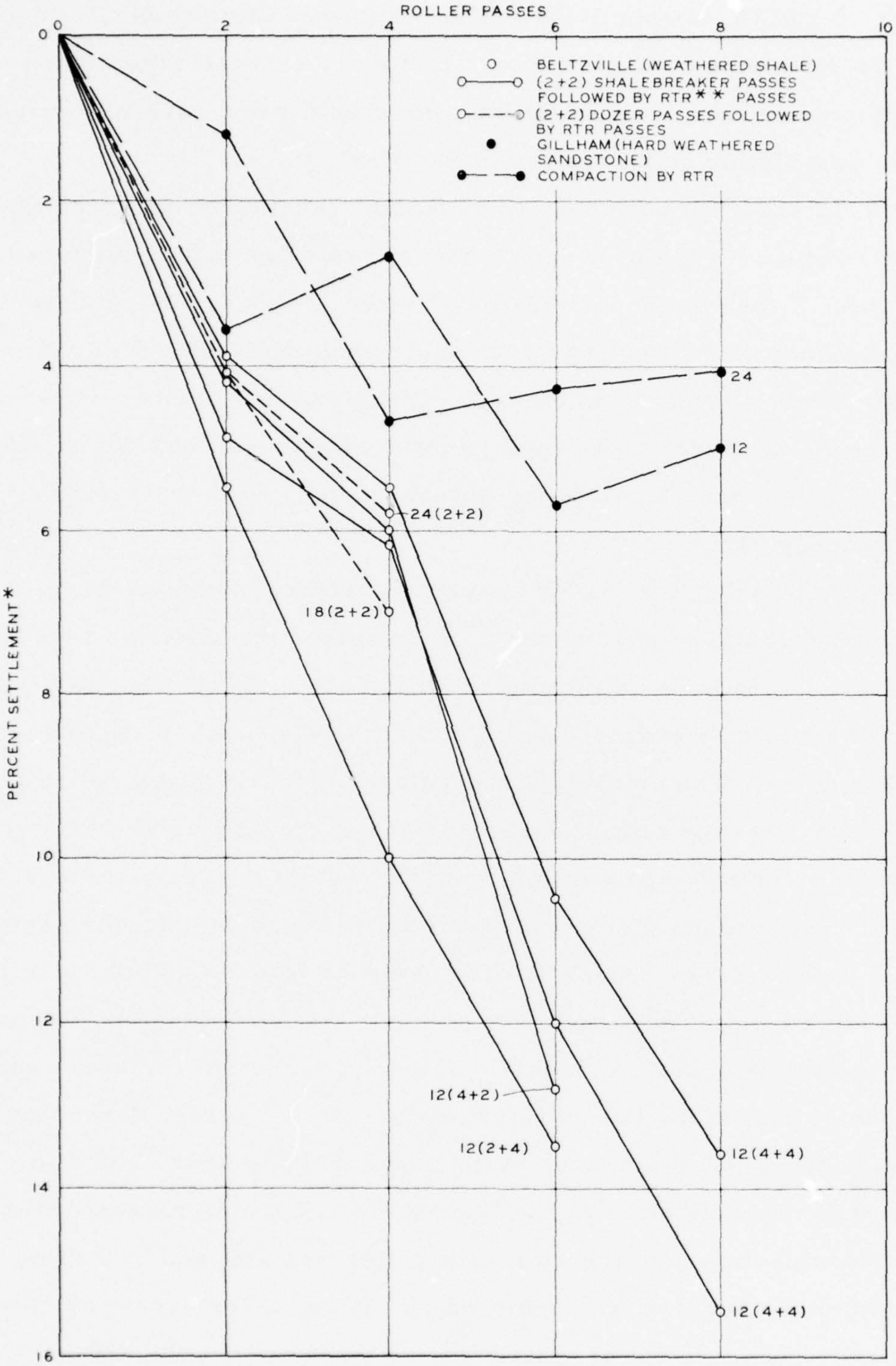


Fig. 60. Summary of percent settlement vs passes by 10-ton vibratory roller



* PERCENT OF ACTUAL LOOSE LIFT THICKNESS
** 50-TON RUBBER-TIRED ROLLER

Fig. 61. Summary of percent settlement vs passes by rollers other than vibratory

tables 10 and 11, respectively. The purpose of these tables is to give concise reference on these items. For more detailed information on any particular part of these tables, one should refer back to the appropriate description given earlier in this Part.

84. As has been previously indicated, the manner in which the test fill is conducted and the type of measurements and tests performed are governed by the type of information desired. This is evident from the varied nature of the test fills previously examined. However, even though there is variance in the type of fills and the methods used to conduct them, there are several general conclusions that can be drawn. These conclusions, along with a general summarization, are given in the following paragraphs.

85. The vibratory roller appears consistently to give the best results for compacting hard rock. In addition, the superiority of the vibratory roller over other rollers (pneumatic, crawler tractor, etc.) seems to increase with increasing lift thickness; i.e., it compacts to a greater depth. The primary disadvantage of the vibratory roller is the amount of breakage caused on the surface of the fill. This results in a surficial buildup of fines, which, in turn, results in a smooth lift surface and leads to stratification within the lifts and poor bonding between lifts. In this respect, there is also evidence that the vibratory roller is more effective on clean rock (say plus 3- or 4-in. material) than on quarry-run material. Reasonable values of loose-lift thickness and number of passes for the vibratory roller is, of course, dependent on the rock type, but seem to be in the range of 18 to 36 in. and 4 to 6 passes, respectively. At the Laurel test fill it was reported that better compaction with the vibratory roller was attained by rolling in the forward direction than was achieved by rolling in the backward direction. This is because the roller tested was so made up that the maximum

Table 10

Summary of Variables Evaluated

Project	Rock Type	Roller	Lift Thickness in in. (Number of Lifts) Test Section A	B	C	D	Total Number of Passes on Each Lift (Each Roller) Test Section A	B	C	D
Beltzville	Partly weathered shale	Shalebreaker followed by 50-ton rubber-tired	12(4)	12(4)	12(4)	12(4)	6(2 + 4)*	6(4 + 2)	8(4 + 4)	8(4 + 4)
	Relatively unweathered shale	10-ton vibratory	18(3)	12(4)			4	4		
	Relatively unweathered shale	*AC HD-20 bulldozer followed by 10-ton* vibratory	24(2)	18(3)			4(2 + 2)**	4(2 + 2)		
Laurel	Hard sandstone	10-ton vibratory	24(1)	36(2)	48(1)	60(1)	8	8 and 10	10	8
	Hard sandstone (plus 3 in.)	10-ton vibratory	48(1)				12			
Gathright	Hard limestone	10-ton vibratory	18(2)				6			
	Hard limestone (plus 2.5 in.)	10-ton vibratory	18(3)	24(4)			6	6		
Cougar	Fine-grained basalt	50-ton rubber-tired	18(6)				4			
	Fine-grained basalt	5-ton vibratory	18(6)				4			
	Medium-grained basalt	5-ton vibratory	18(1)	24(1)	36(1)		6	6	6	
	Medium-grained basalt	10-ton vibratory	18(1)	24(1)	36(1)		6	6	6	
	Medium-grained basalt	5-ton vibratory followed by 10-ton vibratory	18(1)	24(1)	36(1)		12(6 + 6)***	12(6 + 6)	12(6 + 6)	12(6 + 6)
	Medium-grained basalt (minus 12 in.)	10-ton vibratory	18(1)	24(1)	30(1)	36(1)	6	6	6	6
New Melones	Weathered metasandstone (6 to 18 in.)	10-ton vibratory	18(4)				6			
	Weathered metasandstone (minus 6 in.)	10-ton vibratory	12(6)				6			
	Fresh metasandstone (6 to 18 in.)	10-ton vibratory	18(4)				6			
	Fresh metasandstone (minus 6 in.)	10-ton vibratory	12(6)				6			
Gillham	Weathered sandstone	10-ton vibratory	12(8)	24(4)			8	8		
	Weathered sandstone	50-ton rubber-tired	12(8)	12(3)	24(4)		8	8	8	
	Fresh sandstone	10-ton vibratory	18(3)				8			
	Fresh sandstone (plus 3 in.)	10-ton vibratory	18(3)	36(3)			8	8		

* Shalebreaker passes followed by rubber-tired roller passes
** Dozer passes followed by vibratory roller passes
*** 5-ton vibratory passes followed by 10-ton vibratory passes

Table 11

Summary of Spreading, Towing, and Compaction Equipment Used

Test Fill	Spreading (S) and/or Towing (T) Equipment	Towing Speed mph	Compaction Equipment Type	Make and Model	Data
Beltzville	Allis-Chalmers HD-20 Bulldozer (S, T)	3	Shale-breaker	Ferguson-Gebhard Model 22	Chisel tips with 1.5-sq-in face area, 3750-psi tip pressure
		3	50-ton rubber-tired roller (RTR)	Bros 50-ton	Four pneumatic tires abreast, 25,000-lb wheel load
		1.5	10-ton steel wheel vibratory roller (VR)	Ferguson Model 230	23,500-lb static weight; operating frequency of vibration (OFV) varied from 1100 to 1300 VPM
			Bulldozer	Allis-Chalmers HD-20	Not given
Laurel	Caterpillar D-6 Bulldozer (S)				
	Caterpillar D-8E and D-8H Bulldozers (S, T)	1	10-ton VR	Ferguson Model 230	23,500-lb static weight; OFV = 1500 VPM
Gathright	Caterpillar D-8 Bulldozer (S, T)	1.25	10-ton VR	Not given	OFV = 1375 VPM
Cougar	Caterpillar D-8 Bulldozer (S)				
	Caterpillar D-9 Bulldozer (T)	3	50-ton RTR	TAMPO	
	15-ton Euclid Rubber-tired Tractor (T)	1.5-2.0	5-ton VR	Bros Model VP-9D	9,950-lb static weight; OFV = 1300 VPM
	15-ton Euclid Rubber-tired Tractor (T)	1.5-2.0	10-ton VR	Bros Model VP-20D	18,900-lb static weight; OFV = 1300 VPM
New Melones	Caterpillar D-8 Bulldozer (S)				
	Caterpillar D-6 Bulldozer (T)	1.0-1.5	10-ton VR	Bros Model VP-20D	18,900-lb static weight; OFV = 1350 VPM
Gillham	Caterpillar D-8 Bulldozer (S)				
	Caterpillar D-7 Bulldozer (T)	1.0-1.5	50-ton RTR	Ferguson	
	Caterpillar D-7 Bulldozer (T)	1.0-1.5	10-ton VR	Bros Model VP-20D	18,900-lb static weight; OFV not given

dynamic force was obtained when the roller was operating in the forward direction. This is probably true of most vibratory rollers.

86. Of the variables evaluated in these test fills, probably the most often studied variable was the amount of rock breakage or degradation. This is because the amount of degradation either directly or indirectly affects the shear strength, compressibility, and permeability of the resulting fill. As previously stated, the vibratory roller causes substantial surficial breakage, especially to soft to medium-hard rock. Noted also was the fact that hauling, dumping, and spreading often caused considerable breakage. It appears that in most cases, regardless of the rock type, smaller size particles were more easily broken (in many cases reduced to dust) than were the larger sizes. Screening was therefore often used to restrict the smaller sizes where breakage was a problem. Generally, this resulted in a better fill with less stratification and better lift-to-lift bonding. The amount of degradation suffered by the rock in these test fills was visually assessed by observation of the rolling operation and inspection of trench sidewalls. A more quantitative assessment was made in some cases by comparing gradations run before spreading and after compaction. Stratification and/or segregation was investigated by visual examination only, as all after-compaction gradations were representative of total lift thickness and therefore did not delineate between zones of finer or coarser material within the lift. An effort was made to determine if the Los Angeles abrasion test could be used as an indicator of the amount of breakage one could expect from a particular type rock; the results of this effort are presented in Appendix B.

87. In situ density tests in rock fill are, by necessity, of a large volume because of the large individual particle sizes involved. It was recognized by most that these tests are time-consuming, expensive,

and yield uncertain results. They were used, therefore, to supplement the settlement data rather than as the primary means to judge compaction. These tests were most often carried out by lining the density pit with a thin plastic membrane and using water to measure the volume. Pit dimensions ranged from 15 by 15 in. in soft rock that broke to small sizes to 6 by 6 ft in larger rock. The 6- by 6-ft (or 6-ft diameter) pit was most often employed. Density determinations by the nuclear method were made at one project (Beltzville) and gave questionable results.

88. With only a very few exceptions, settlement readings were used to judge the effectiveness of the roller in achieving compression of the rock fill. In order to be representative, a grid composed of many points (usually on 5- or 6-ft centers) was established on the surface of each lift. Level readings were taken at appropriate intervals (usually after every two passes) and the settlement of the entire lift was computed by averaging the readings from all grid points for each interval of rolling. There were problems in some cases in getting representative readings due to rotation or shifting of individual surface rock at the point of the reading. The steel plate used at Cougar (fig. 27) appeared to be very helpfull in obtaining representative readings at a given point. In order to obtain reasonable settlement data, most districts threw out those points that showed heave after compaction when all others around it showed compression, and these were not included in the averaging process. The settlement data were plotted as accumulative settlement (generally as a percentage of loose lift thickness) versus number of roller passes and loose lift thickness. These curves usually enabled an adequate comparison of the variables being evaluated to be made and, in most cases, gave a rough idea of the optimum number of passes and/or loose-lift thickness.

Summary of Rock Test Fill Data from 14 CE Projects

89. Table 12 contains a tabular summary of data available from 14 CE rock test fills. The six test fills previously analyzed are given along with eight more. The purpose of this compilation is to provide a reference list in order that a designer might be able to obtain additional information from those CE districts that have had previous experience with test fills similar to the one he is designing. The information given is brief and is intended to present only the most general parameters of a project. For further information, the districts themselves should be contacted.

Table 12

Summary of Data

			Rock Data					Compaction Data				Density Data					
Dam and Location	Division and District	Source of Data	Types	Particle Shape Before Rolling	Breakage During Rolling	Gradations	Los Angeles Abrasion Tests	Type of Rollers	Speed of Rolling mph	Uncompacted Lift Thickness in.	Number of Passes	Test Method	Number of Tests	Average Dry Density pcf	Photographs	Settlement Data	Foundation Underlying Test Fill
Raystown Juniata River, Penn.	NAD Baltimore	Brief summary from Dist. and DM #8, vol. II, Nov., 1966	Sandstone (SS), Siltstone (Si), Shale (Sh) (Various mixtures of the above were used as well as each type alone)	(SS)-slabby (Si)- blocky (Sh)-blocky	SS broke into blocky shape, Si and Sh broke into smaller blocky shape plus numerous fines	None before rolling. One for each density test	None	D-9 dozer 10T vibratory 50T pneumatic 32T Euclid truck	Not given	8, 12, 18, 24, & 36	Varied from 2 to 8	6' dia ring with plastic sheeting and water	10 9 1 2 2	118-145 111-137 130 128&130 130&137	Fill surfaces before and after compaction and sides of test trenches	Settlement vs number of passes and settlement vs layer thickness	Rock
Beltzville Leigh River, Penn.	NAD Philadelphia	Brief summary from Dist. and Supp. #1 to DM #11	Partly weathered shale (wSh); hard, dense unweathered shale (Sh)	Both elongated	Limited to extensive, depending on type roller used	Before and after rolling, 3 on wSh, 2 on Sh	1 on wSh & 1 on Sh	Shale breaker (3750 psi) 50T pneumatic 10T vibratory AC HD-20 crawler tractor	1.5 to 3.0 (estimated)	12, 18, & 24	Varied from 2 to 8	3' × 3' wooden template with sand or plastic sheet and water	6 1	122 127	Fill surfaces before and after compaction and sides of test trenches	Settlement vs number of passes	Not described but foundation settlement measured
Gathright Jackson River, Va.	NAD Norfolk	Brief summary from Dist. and rock test fill report	Mod. hard to hard sandy limestone with shale lamina and calcite layers	Angular	Minor	1 after rolling, none before rolling	3	10T vibratory	1.0 to 1.25	18 & 24	6	3' × 3' wooden template with plastic liner and water	2 in each lane	18" lift-142 24" lift-118	Fill surfaces at various stages of compaction.	Settlement vs number of passes	Not described but foundation settlement measured
Cougar South Fork McKenzie River, Oregon	NPD Portland	Summary from Dist. and paper by Bertram (2 test fills, 1961 and 1962)	Fine-grained glassy basalt (fB), medium-grained vesicular basalt (mB)	fB-angular, mB-angular and blocky	fB-considerable fines produced mB-less fines produced than the fB	None before rolling. Several after rolling	1	1961: 50T pneumatic, Exp. impact, and 5T vibratory 1962: 5T vibratory, 10T vibratory	3 3 2 Est. at 2	1961: 18 1962: 18, 24, 30, & 36	1961: 4 1962: 2, 4, & 6	1961: One cu yd vol with plastic sheeting & water 1962: Not given	1961: 3 3 3 1962: 1	116 109 119 113	Fill surfaces before and after compaction and sides of test trench	Settlement vs lift thickness for 2, 4, and 6 passes (1962 only)	Not described
Fall Creek Fall Creek, Oregon	NPD Portland	Summary from Dist. and report on test fill (2 test fills, 1962 and 1965)	Tough massive basalt (Porphyritic and amygdaloida' phases)	Blocky and angular	Moderate	None before rolling. Several after rolling	None on quarry samples, 3 on core samples	1962: D-8 dozer 1965: 10T vibratory & 15T vibratory	Not known < 1.5	1962: 30, 36, & 60 1965: 48	1962: 2 1965: 2 to 6 (last lift-11 passes to observe breakage)	← 1962, No tests performed → ← 1965, No tests performed →			1962: Fill surface prior to compaction and sides of test trenches 1965: Fill surface after compaction	1962 and 1965: Settlement vs number of passes	1962: Weathered rock 1965: Not described
Laurel Cumberland River, Ky.	ORD Nashville	Brief summary from Dist. and DM #3 Part B supplement, Jan. 1964 (2 test fills reported, but only test fill No. 2 considered here)	Hard sandstone (hSS) and soft sandstone (sSS)	hSS-pyramidal, cubical, and wedge-like, sSS-not given	hSS considerable fines produced; sSS-broke down readily into tight, compact mass	None	4	hSS-10T vibratory sSS-23T sheepsfoot roller	Not given	24, 36, & 60 9 & 12	Rolled until less than 1% settlement	sSS: Sand cone and 1/30 cy push cylinder hSS: None	sSS: 1 after every 2 passes of roller	96 to 116	Fill surface prior to and after compaction and sides of test trenches	Settlement vs number of passes and layer thickness	Rock

Table 12 (Continued)

Summary of Data

Dam and Location	Division and District	Source of Data	Rock Data: Types	Rock Data: Particle Shape Before Rolling	Rock Data: Breakage During Rolling	Rock Data: Gradations	Rock Data: Los Angeles Abrasion Tests	Compaction Data: Type of Rollers	Compaction Data: Speed of Rolling, mph	Compaction Data: Uncompacted Lift Thickness in.	Compaction Data: Number of Passes	Density Data: Test Method	Density Data: Number of Tests	Density Data: Average Dry Density pcf	Photographs	Settlement Data	Foundation Underlying Test Fill
Paint Creek Paint Creek, Ohio	ORD Huntington	Test Embankment Construction Report, Jan. 1968	Dolomite	Not given	Not given	None before rolling, 2 after rolling	2 (26% loss at 500 revs)	Loaded truck (28 ton) 60 psi tire pressure	Not given	12 & 24	4	4' × 4' × 4' pit lined with plastic; water used for volume	2	117 (24" lift) 130 (12" lift)	Equipment, side slopes, test pit, and quarry	No settlement data taken	Stockpile rock (35' thick)
Carters Coosawatee River, Ga.	SAD Mobile	Brief summary from Dist. on rock placed in dam, ASCE Journal of Const., Div, vol. 92 #CO3 Sept. 1966, and DM #8, 18 Sept. 1964 (5 test fills)	Dam: Zone 3A - Fresh quartzite Zone 3B - Slightly weathered quartzite Zone 3C - Moderately weathered quarry run random	Angular Angular Angular	Minor Minor Minor	One before rolling and one after rolling. Source of samples not given	3	10T vibratory 12.5T vibratory	<1.5 <1.5	3A - 36 3B - 24 3C - 18 & 24	10T - 4 12.5T - 2	6' dia ring with plastic sheeting and water, 1 cu yd	3A - 49 3B - 12 3C - 25	136 140 140	Zone 3A fill surface before and after rolling	None	Rock
			Test fills: #1 Fresh quartzite #2 Phyllite & argillite #3 Mod. weathered phyllite & argillite #4 Highly weathered rock #5 Fresh quartzite	#1 Blocky #2 Blocky to slabby #3 Blocky to slabby #4 -- #5 Blocky	#1 Minor #2 Considerable #3 Considerable #4 Complete #5 Minor	None	None	#1 & 2: 5T vibratory #3: Sheepsfoot, 50T pneumatic, 5T vibratory #4: Sheepsfoot #5: 10T vibratory	Not given	#1 & 2: 18 & 30 #3: 12 #4: 6 #5: 12, 18, 24, 36, 48, 60	#1 & 2: 4 & 8 #3: 4 & 8 #4: 6 #5: Up to 22	← No tests performed →			None	#1 & 2: Settlement vs number of passes and lift thickness #3: None #4: None #5: Settlement vs number of passes and lift thickness	#1: Rock #2: Sand #3: Weathered rock #4: Clay & weathered rock #5: Natural ground (clay?)
New Hope Cape Fear River Basin, N. C.	SAD Savannah	Brief summary from Dist. and DM No. 9, vol. I, Supp. #2, Aug. 1967	Weathered basalt	Angular to subangular	Particles tend to become more subangular and fines produced	None before rolling. After rolling, one for each density test	3, but performed on sample after compaction	10T vibratory	1.5	15 & 24	2, 4, & 6	6' dia ring with plastic sheeting and water	6	143 to 151	Fill surface before and after compaction and 1 photo of side of test trench	Table giving reduction in lift thickness during compaction	Not described
New Hogan	SPD Sacramento	Office report, Dec. 1962	Two types: #1 Volcanic agglomerate with embedded angular fragments of andesite #2 Volcanic tuff	Subangular Subangular	Protruding rock and corners of large rock broken by impact, some shattering of medium rock on surface	None	1	10T vibratory 50T pneumatic	1 to 1.5 3	24	2, 4, & 6 2, 4, & 6	← No density tests →			Fill surface after compaction and sides of test trenches	Settlement vs number of passes	Weathered rock and clay
New Melones Stanislaus River, Calif.	SPD Sacramento	Data for Consultant Board Meeting, 16-18 June 1969	Meta-volcanic: weathered (w) fresh (f)	Angular to blocky	Protruding corners broken, some shattering of medium sized rock to gravel sizes	Several before and after rolling	2 w: 20% loss f: 15% loss	10T vibratory	1.5	12, & 18	6	← No tests performed →			Equipment, fill surfaces, inspection trenches, typical rock	Percent settlement vs number of roller passes	Sound rock

Table 12 (Concluded)

Summary of Data

Dam and Location	Division and District	Source of Data	Rock Data: Types	Rock Data: Particle Shape Before Rolling	Rock Data: Breakage During Rolling	Rock Data: Gradations	Rock Data: Los Angeles Abrasion Tests	Compaction Data: Type of Rollers	Compaction Data: Speed of Rolling mph	Compaction Data: Uncompacted Lift Thickness in.	Compaction Data: Number of Passes	Density Data: Test Method	Density Data: Number of Tests	Density Data: Average Dry Density pcf	Photographs	Settlement Data	Foundation Underlying Test Fill
Gillham Cossatot River, Ark.	SWL Tulsa	*Test Embankment* Report, August 1964	Weathered sandstone (wSS), fresh sandstone (SS)	Blocky	Moderate	2 before rolling, 6 after rolling	None	wSS: 50T pneumatic SS: 10T vibratory	<1.5	wSS: 12 & 24 SS: 18 & 36	8	6' × 6' wooden template guide for pit volume by hand measurements	wSS: 3 SS: 3	135, 128, 137 91, 83, 104	Fill surfaces, equipment, inspection trenches, and quarry	Percent consolidation after every 2 passes	Firm shale
Stockton Sac River, Mo.	MRD Kansas City	Report-Rolled Rock Test Fills, April 1970	Dolomite-limestone (DLs) and limestone (Ls)	Not given	Moderate	Before and after rolling on all type rock	None on DLs 10 on Ls % loss varied from 29.5 to 43.5)	10T vibratory	1.5	24, 36, 24, & 48	6 to 10	Pre-dug pit lined with heavy plastic. Volume approx 5 cy	DLs: 3 Ls: 1	128, 116, 124 123	Equipment, quarry fill surfaces, density pits, and inspection trenches	Settlement vs number of passes and lift thickness	Existing rock fill for dam (test fill was conducted concurrent with dam construction)
Fukuji Fukuji River, Okinawa	POD Okinawa	Report on Test Fill Sections 4 and 5, vols 1 and 2, Nov 1969	Limestone (material was separated for compaction as +6" and −6"	Angular and wedge-shaped	Minor	Several before and after rolling	Several on −1" material (avg loss = 35%)	10T vibratory (for +6") 50T pneumatic (for −6")	1.36 1.03	12, 18, 24	2, 4, & 8	Sand cone* volume on −6" material, no tests on +6" material	*Sand observed flowing into voids of fill, therefore results are considered inaccurate		Equipment, quarry, typical rock, fill surfaces, inspection trenches	Settlement vs lift thickness	Rock

PART III: RECOMMENDED TEST FILL PROCEDURES

Introduction

90. This Part is basically a synopsis of those procedures employed in the test fills previously discussed in Part II that appear to have been the most effective. Included are general considerations, planning and design, construction and execution, and measurements and observations of rock test fills. In addition, some discussion is given on the general aspects of analyzing the test data.

91. The design of a rock test fill is, of course, dependent on the objectives to be achieved. In turn, the objectives are usually primarily dependent on the rock type. Hence some of the procedures to be employed for a particular test fill are established solely on the basis of the type of rock to be used. For this reason, this part is not comprehensive in its treatment of any specific item, but instead presents more general guidelines for a thorough test fill program. In addition, a few of the common problems encountered during construction of the test fills presented in Part II are given in the discussion of the procedures to which they apply.

General Considerations

92. It is often necessary to construct test embankments of rock to ascertain just how the rock will behave during placement and compaction, and how it will function as an engineering material thereafter. The main properties of interest of the compacted rock fall under or relate to one or more of the following three categories: (a) shear strength,

(b) compressibility, and (c) permeability. The controlled variables involved in a rock fill that affect these three properties are: (a) type of equipment, (b) number of passes and loose lift thickness, and (c) initial rock gradation. To determine the effect of these controlled variables on the shear strength, compressibility, and/or permeability, qualitative and quantitative assessments are usually made of (a) stratification and/or segregation, (b) density, and (c) lift-to-lift bonding of the in-place material. The one item which either directly or indirectly affects all of the properties of the in-place material is the amount of degradation or breakage the rock suffers in the process of removing it from its natural in situ position and compacting it in the fill. Therefore the test fill program must be closely tied to the quarry operation. This may include the operation of a screening plant (or grizzly) or selective loading procedures in addition to the normal quarrying procedures.

93. A most important item for consideration is that procedures involved in constructing a test fill must simulate, as closely as possible, feasible construction procedures to be used in the prototype fill. This requires some experience in the construction of rock fills. If test fill procedures do not closely simulate actual construction procedures, the value of the test fill is reduced and may even do more harm than good.

94. Test fills are normally conducted either before construction begins (i.e., at some time during design stage) or provisions are made in the bid documents to allow for their construction during the early phases of actual construction. Past experience has demonstrated that often much more rock had to be wasted than was anticipated to obtain rock suitable as a fill material. Thus, for fear of not being able to obtain representative rock (i.e., rock representative of what will be used in actual construction) without considerable extra expense, some test fill programs are scheduled during the early phases of construction.

Another advantage of this is that equipment to be used in actual construction is available for the test fill. On the other hand, the advantages of a prebid test fill include (a) results can be used by the designer to prepare specifications for rock placement and compaction (and blasting if a test quarry was also conducted), and (b) the quarry face can be inspected by prospective bidders. Therefore a properly conducted prebid test fill will most likely result in a lower bid; but if representative rock or equipment procedures are not used, the final cost of the rock fill may be higher as a result of changes that must be made after the contract has been let and construction begun. The decision of when to conduct the test fill, then, is one which must be based on features of the individual project.

95. A test fill program must be flexible. Due to natural rock variations and unpredictable behavioral characteristics, it is often impossible to come up with a definite program from which there are no deviations. Often procedures and specifications must be altered based on results from completed portions of the program. This was made evident in studying the test fills reported in Part II. The use of an equipment-rental type of contract, such as employed at the Laurel test fill, allowed the project a great deal of flexibility.

Planning and Design of Test Fills

Location

96. The test fill should be located as near the test quarry or rock source as possible. This will ensure economy of operation and help prevent any necessity for stockpiling, which is disadvantageous from the standpoint of segregation. The test fill site should be as level as possible, and of sufficient size to accommodate the fill area and to ensure

that full equipment mobility is possible. In addition, provisions should be taken to ensure that the fill area is well drained.

Geometry

97. There is a wide variety of possible rock test fill configurations, the choice of which depends on the objectives of the test program and the variability and availability of rock. Hence, the following discussion is of a general nature, applicable to rock test fills in general.

98. The test fill should be of sufficient size to allow its performance to be as close to the actual fill behavior as possible. This means the effects of scale should be minimized as much as possible. Widths and lengths of individual test sections should be of sufficient magnitude so that settlement readings reflect settlement from compactive effort alone and do not include any settlement resulting from lateral bulging of the fill. For the test fills given in Part II, individual fill sections ranged in width from 18 to 70 ft and in length from 50 to 125 ft. For most test fills, a width of 30 to 50 ft should be adequate with the length at least equal to the width but 20 to 30 ft longer, if possible. Generally, the individual fill sections abut each other longitudinally. In some cases, a fairly level surface grade is maintained by filling in between the test sections with random material; while in other cases, no filling is done and each section is more or less an independent fill (see figs. 31 and 45). It would seem the configuration used is primarily a matter of preference, although by maintaining a level top grade, the equipment would have a little more mobility and any bulging of the side slopes would be minimized. However, if the equipment is to be operated in the longitudinal direction only and the sections are wide enough to begin with, then these advantages disappear.

Test sections

99. As a rule, an individual section of test fill should not contain

different materials or be composed of different lift thicknesses or of lifts compacted by different equipment; nor should a different total number of passes be applied to succeeding lifts. For example, suppose it is desired to evaluate 18- and 36-in. lifts. It would be more desirable to have two fill sections, one containing 18-in. lifts only and the other 36-in. lifts only, rather than to have one section containing lifts of both thicknesses (e.g., three 36-in. lifts overlying three 18-in. lifts). If for some reason, however, only one test section is to be built, measures should be taken either to eliminate or to determine settlement of the underlying lifts resulting from rolling the overlying lifts. This could be accomplished by either (a) installing settlement plates on the top of the last lift of the lower group, or (b) rolling the last lift (or each lift) of the lower group until no further settlement occurs. Settlement plates would allow settlement in the lower lifts to be subtracted from the measured settlement resulting from rolling the upper lifts. This would also give an idea of just how much additional compaction underlying lifts do, in fact, receive when overlying lifts are rolled. However, there are no records of settlement plates having been used for this purpose (i.e., used other than to measure foundation settlement) among the data gathered for this report. Of course, there is always the problem of getting good settlement data from plates, and much care must be taken in installing the plates and bringing the pipes up through each successive lift. Rolling until no further settlement occurs is disadvantageous if the number of passes exceeds a practicable compaction procedure in that it compromises the value of the inspection trench since extended rolling will no doubt alter the fill characteristics. Therefore, if at all possible, separate test sections should be used for each variable to be evaluated. In any case enough lifts should be used so that a good average settlement curve can be obtained for all like lifts in each zone. This will allow a

meaningful comparison to be made. Normally, four to five lifts are sufficient. The New Melones test fill is a good example of this in that four different test sections of four to six lifts each were used to evaluate two different lift thicknesses composed of two different materials.

100. Side slopes of test sections are generally on the order of 1V on 2H , and longitudinal ramp slopes range from 1V on 5H to 1V on 10H. Ramp slopes should be flat enough to ensure ease of operation of the equipment, and thus would depend somewhat on the type of equipment specified.

Equipment

101. Generally, loading and hauling equipment should be used that will result in the most efficient operation and which might be expected to be used for the actual construction. Crawler tractors are generally used for spreading; although in some cases where such tractors would cause excessive breakage of the rock, rubber-tired equipment is employed. It is often desired to compare results of compacting with several types of rollers. Ten- and 15-ton vibratory rollers and 50-ton pneumatic rollers are most often used, with crawler tractors being specified to a lesser degree. Many of the test programs described in Part II had the evaluation of a vibratory roller as one objective. This is because they were conducted at a time when the vibratory roller was fairly new (the first 10-ton vibratory roller was tested at the Cougar project in 1962) and its performance was unknown. The vibratory roller has now been proven to be a very effective means of compacting rock, especially for thicker ($\geq$ 24 in.) lifts of hard rock. Therefore, future test fills of hard rock will probably be less concerned with evaluation of various types of rollers than were past test fills; although new and larger self-propelled units may maintain roller evaluation as a primary variable. On the other hand, test fills of friable or weathered

material where it is desired to break the rock down often employ compaction equipment such as pneumatic or tamping rollers for evaluation.

Construction

Foundation preparation

102. The proper preparation of the foundation for the test fill is of special importance since settlement readings on the fill are commonly used to evaluate the compaction obtained. Fortunately in areas near quarry sites, rock foundations can usually be provided with a minimum of overburden stripping. If, however, the foundation consists of soil or weathered rock, it must be thoroughly compacted prior to fill placement, preferably until no further significant settlement can be observed. Where further consolidation of a compressible foundation under fill loads is possible, settlement plates should be installed in the foundation to provide data needed to correct the test fill settlement readings (see Gathright test fill description). A thoroughly compacted rock pad (or leveling course), 2 to 3 ft thick, should be placed on the foundation (whether soil or rock) prior to placing the first test lift in order to ensure that all foundation depressions and undulations are filled and a level surface results. Material for the pad could be either the same rock to be used in the fill or rock obtained prior to getting into representative material that would have to be wasted anyway. Placement should be in at least two lifts and it should be compacted until negligible settlements are observed from level readings made on its surface.

Fill placement

103. Placement of rock to the desired loose lift thickness must simulate expected construction placement procedures. Generally, for hard rock fills, placement is also done in the manner that will minimize

segregation. Segregation may adversely affect the permeability of the fill and can prevent optimum compaction, which may result in increased compressibility and reduced shear strength. To lessen the likelihood of segregation during placement, the work required to get the rock to the desired loose lift thickness should be kept to a minimum. This involves dumping the rock as close as possible to the position it is needed. The usual practice is to dump at the leading (advancing) edge of the lift being placed and spread from there with a dozer (see para 100). The loose lift thickness is usually checked by level readings. One pass of the vibratory roller with the vibratory unit off is usually made prior to taking the initial level readings, in order to smooth the surface to facilitate marking of the level grid and making the initial level readings.

104. Many of the projects in Part II reported considerable breakage caused by the crawler tractor during spreading. To keep this breakage to a minimum, the amount of coverage by the tractor should be kept to a minimum. When the rock-fill material is particularly susceptible to degradation, only rubber-tired equipment can be specified to work on the fill or the grousers or cleats of the tractors can be covered with rubber pads to simulate rubber-tired equipment. Even though covering the grousers with rubber pads will not exactly simulate actual placement procedures, it is an expedient means to determine if there is a difference in material degradation after being worked with tracked equipment and with rubber-tired equipment. In any case, where breakage is to be kept to a minimum, the material qualities will suffer less under the least amount of working.

105. Rock larger than the maximum size permitted is termed oversize. The maximum rock size should not exceed about 0.9 of the loose lift thickness. Oversize particles are generally thought to impede compaction because the roller rides up over them, leaving the surrounding

material virtually uncompacted. This is certainly true of rock in impervious fills, but it was reported in some cases that rocks which protruded above the normal lift surface (presumably oversize) were either crushed by the roller or punched down into the underlying lift. In any event, oversize material is presently not allowed in the fill. It is usually best to attempt removal in the quarry either by use of a grizzly or by selective loading procedures, but small quantities of oversize rock in the fill can simply be shoved to the side of the test lane.

Compaction

106. After the rock has been placed to the desired loose lift thickness, the compaction operation is begun. As stated previously, it is important that this be accomplished in such a manner as to simulate anticipated construction procedure, except for interruptions required to make measurements and observations. Normal procedure is to tow the roller in the forward direction across the test fill, with the equipment turning around after each pass. Usual speeds for vibratory and rubber-tired rollers are 1 to 1.5 mph and $\pm$ 3 mph, respectively. The number of passes to be made may be specified (as 4, 6, 8) or a maximum number can be specified. In the latter case, the maximum number of passes specified should be such as to ensure that a reasonable number of passes for actual construction is reached and surpassed. In some cases, this has been achieved by not setting a specific maximum number but by requiring that rolling is to be discontinued when only a small amount of settlement is produced by a pass (1 percent was used at the Laurel project). The disadvantage of doing this has been previously discussed in paragraph 99.

107. After compaction of a given lift has been completed and all tests and measurements have been made, the surface of the completed lift should be covered with a marker material such as lime or a heavy

plastic membrane (8 mil maximum thickness). This will facilitate identification of individual lifts when an inspection trench or test pit is excavated so that the distribution of fines, stratification, and lift-to-lift bonding can be assessed by both visual examination and testing of samples taken from the trench sidewalls.

108. The most common problem that arose during compaction of the test fills reviewed in Part II was particle breakage on the lift surface due to roller action, which caused surficial buildup of fines. As a result of this breakage, several undesirable characteristics were reported: (a) the compactive effort reaching the lower part of the lift was dampened, resulting in the lower part of the lift being less compact than the upper; (b) a stratified section resulted; and (c) lift-to-lift bonding was poor. The amount of breakage caused by the roller appeared to be primarily a function of its static weight; i.e., the 10-ton roller caused more breakage than did the 5-ton, the 15-ton more than the 10-ton, etc. Hence, there is not much that can be done with a particular roller to reduce surface breakage. The problem must be attacked by varying the rock gradation and using the practices previously mentioned to reduce breakage in the spreading operation.

Measurements and Observations

General

109. Both measurements and visual observations are of importance, since the overall conclusions reached from the results of a test fill are as much qualitative as quantitative. The importance of good diary keeping and photographic records cannot be overemphasized, especially in view of the fact that design personnel who are to use the information usually cannot be present at the site at all times. Like the layout and

design of test fills, the measurements and observations made are highly dependent on the primary objectives of the fill program. However, most of the following items should be a necessary part of a test fill program even though many of them are difficult and expensive.

110. Advance planning and scheduling of tests are an integral part of the overall design. In this respect, flexibility is also important, as only rarely can the test program be fully laid out beforehand and carried out with no deviations. Provisions should be made for supplemental tests and for relocation, if necessary, of test sites. Personnel who are to conduct the tests should be made familiar with the program and procedures as, unlike laboratory and more common field tests, no standards exist for most of the necessary tests and measurements. Personnel should also be made aware of what is expected of them as far as visual observations are concerned. It is desirable for someone from the design group to be present at all times.

111. Detailed procedures for the various tests and measurements described herein are not given. For information of this type, it is suggested that those districts who have had experience with the particular type of test required be contacted.

Densification

112. General. The densification of rock fill is usually judged by (a) measuring the settlement resulting from compaction, (b) performing in situ density tests, (c) detailed observation of inspection trenches, and (d) a combination of the preceding items. Due to the difficulty and expense of making enough tests to ensure representative results and because sometimes results of the in situ density tests are questionable, such tests should not be used as the sole means of judging the effectiveness of the compaction process. Settlement determination by methods subsequently described should be used for this purpose in conjunction

with visual observations in inspection trenches and with in situ density tests when available. In situ density tests are useful in that they provide quantitative values and allow comparison of the densities of different material to be made.

113. Settlement. Settlement of the fill surface is measured by taking readings at many points on the test section located in a grid pattern on 5- or 6-ft centers, or closer if the lift surface is very rough. These grid points should be positioned in the central part of the test section with none closer than 5 ft from any side slopes, ramps, or uncompacted zones (see fig. 32). By avoiding edge effects, observations should be more representative of the actual compaction being applied by the roller. There are several methods of establishing the grid. In most cases, wires or strings are pulled from perimeter stakes set at the desired spacing. Another satisfactory method utilized a light-weight template consisting of a metal frame strung with wire or twine. In any event, after the points are located, they should be well marked on the fill surface with contrasting paint to facilitate identification for subsequent level readings.

114. Prior to establishing the grid points on an uncompacted lift surface, a leveling pass should be made by the vibratory roller with the vibratory unit off. This will provide a smoother surface upon which to establish the grid points and from which the loose lift thickness can be determined. This leveling pass with the vibratory roller can also be used when other types of rollers are to be used for compaction. After taking the initial level readings to establish the loose lift thickness, the compaction operation is commenced, with level readings made on the grid points after every two passes of the roller.

115. Since a reading at a point must represent the area surrounding it (for points on 6-ft centers, for instance, each point represents a

6- by 6-ft area), it is important that the level rod be placed where it is indeed representative of this area and not influenced by local irregularities at the point. This is a definite problem on a rough rock surface, and for this reason the device used at the Cougar Dam test fill is recommended. This device, shown in fig. 27, consists of a 1-ft-sq metal plate with a raised button in the center upon which the level rod is placed for readings. A handle made from a steel rod is attached to the plate to facilitate firm seating of the plate and transport of the plate from place to place. The level instrument should be located carefully with regard to the compacting equipment so as to avoid any disturbance if it is to be left in the same position throughout placement and rolling operations. Bench marks should be established in secure places well away from the fill area, and the instrument checked back frequently to these bench marks. Disturbance of the instrument was mentioned in reports on some test fill programs as being a possible source of erratic settlement readings.

116. The settlement of a particular lift under a given number of passes is obtained by averaging the settlements measured at all points in the grid. Those points that indicate heave rather than settlement can be eliminated from the averaging process if none of the surrounding points indicate heave or if it is noted that a local condition at the point is not representative of the surrounding area. If settlement plates have been installed beneath the fill, they should be read along with the grid points, and any settlement thus indicated should be subtracted from the lift settlement.

117. In situ density tests. The volume of material excavated for an in situ density test in a rock fill must necessarily be large so as to minimize the effects of particle size on the results. For rock of the sizes under consideration in this report, a 6-ft-diam hole is normally

used. Because of the likelihood of stratification, the hole should extend through the entire lift being tested. A steel ring or a frame of wood 2-by-4's can be used as a surface template, although the steel ring is preferred because it remains more stationary during the test. The volume of the excavation is determined by measuring the volume of water or sand required to fill the hole after the hole has been carefully fitted with a polyethylene membrane to prevent leakage. Detailed procedures for performing this test are given in Appendix C. Computation of volume by measuring the dimensions of the hole is too inexact and should not be done. When excavating material from inside the hole, care should be exercised to avoid loosening the sidewalls. This usually requires considerable handwork.

Gradation tests

118. Gradation tests are used to determine the amount of breakage the rock has suffered during placement and compaction. This is accomplished by running gradation tests on samples representative of the material as delivered to the fill ("before" gradations) and on samples taken from the compacted fill ("after" gradations). Differences in the two resulting curves are indicative of the breakage. It is important that the before gradation samples be taken from the material delivered to the fill; that is, after any selective loading or other processing procedure has been done. Test quarry shot gradations made to evaluate different blast patterns can be used as before gradations if the material goes directly to the fill as is. After compaction samples are usually obtained from material excavated from density test holes or from the sides of the inspection trenches or test pits; otherwise a hole must be dug in the fill to obtain a gradation sample. Since stratification and/or segregation is a rather common problem with vibratory compacted rock fill, it is important that the entire depth of a lift be sampled (if the gradation is to

represent the entire lift) and not just part of it. Where stratification is evident or suspected, two or more samples, each from a particular zone within the lift, can be taken to help delineate the degree of stratification or the overall distribution of fines with depth.

Percolation tests

119. Percolation tests are run to check the drainage characteristics of the compacted fill. Results from percolation tests can be used to provide a relative comparison of the drainage characteristics of lifts composed of different material or placed and compacted in a different manner. In addition they serve to verify the design assumption that the rock fill is free draining.

120. Percolation tests are normally performed by filling pits excavated in the compacted fill with water and observing the rate of drainage. The pits are usually a minimum of 3-ft square and at least one lift thickness deep. These tests are, by their very nature, more qualitative than quantitative. However, there have been cases where the rate of fall of the water surface in a pit of known dimensions was measured and a crude value of permeability computed therefrom. Another method of performing this test is to insert a 3- to 6-in.-diam steel pipe into the fill and observe the time required for it to empty after filling it to a mark. This method is highly subject to localized fill conditions such as concentrations of fines or voids.

Los Angeles abrasion tests

121. The Los Angeles abrasion test has potential use in predicting the amount of breakage a particular rock type will undergo during handling and compaction. Analysis of available data (see Appendix A) did not give satisfactory correlations, possibly because of variations in test procedures and the limited data on test fill gradations. It is believed, however, that in order to make use of the test to predict degradation

during handling and compaction, the following items must be given consideration:

<u>a</u>. The test be run according to the method given in ASTM C535-65 and on the coarsest gradation allowable (grading 1, 3 in. to 1-1/2 in.).

<u>b</u>. A gradation test be performed on all the material after the test rather than just specifying the results as the percent loss based on the No. 12 sieve.

<u>c</u>. A sufficient number of tests be run so as to make the results meaningful.

<u>Visual observation of placement and compaction operations</u>

122. Because of the nature of a test fill, visual observations of the various construction procedures are important as a source of qualitative supplemental information, and in some cases primary information. Some items that should be closely observed are (a) breakage of the rock during spreading and compaction, (b) segregation during spreading, (c) the smoothness of the surface after each interval of rolling, (d) the amount of vibration felt while standing at various points on the fill as the roller passes by, (e) any variation in the established behavior of any phase of the construction operation, and (f) the appearance of the fill during and after rains. All visual observations should be well documented with photographic evidence and a written record consisting of a detailed diary.

<u>Inspection trenches and test pits</u>

123. It is highly desirable to expose a cross section of each test section in order that the general in situ characteristics of the compacted fill might be observed. This is done by the use of inspection trenches or test pits. The test pit is excavated from the top down through the lift immediately after rolling or through all or a part of the lifts after the entire test section is completed. The inspection trench is a cut made through the entire depth and usually across the full width of the completed

test section. Excavation is usually done with a front-end loader or dozer. The inspection trench is most often used, as the only advantage of the test pit is that it can be dug at any stage during the rolling operation, but this is usually not required. Except as a source of material for gradation tests, the use of the inspection trench is primarily for qualitative examination of the compacted fill.

124. Inspection trenches are usually cut to the width of the dozer blade or front-end loader bucket. In any event they should be of ample width to allow plenty of room for inspection by personnel. Excavation by whatever equipment is used will result in the sidewalls being loose and covered with some fall-out material. This requires that some handwork be performed in the area to be inspected to get a view of representative in situ material. The same holds true if samples are to be taken from the sidewalls.

125. Items of interest when inspecting a pit or trench are (a) stratification, (b) segregation, (c) occurrence of voids, (d) the amount of rock-to-rock contact, (e) bonding between lifts, and (f) the general tightness of the fill, including the stability of the sidewalls. As with all visual observations, good documentation of the inspection trench is very important and should include both photographs and a written record.

Evaluation of Test Results

126. It has been shown that the information accumulated from the construction of a test fill consists of both quantitative and qualitative data. In analyzing the data and drawing conclusions for design purposes, it is necessary to consider all data. That is, qualitative data should not be neglected even in the presence of strong quantitative data.

127. Settlement data are generally the most useful quantitative

information for determining the proper loose lift thickness, number of passes, roller type, and material gradation. Settlement data are normally plotted with settlement (either as actual settlement or in percent of actual loose lift thickness) as the ordinate and number of roller passes or loose lift thickness as the abcissa. The type plot used will, of course, be highly dependent on the variable to be evaluated, but the data should always be plotted in as many ways as possible, as some relations will show up better in some plots than in others. Generally, it is preferable to plot settlement as percent of actual loose lift thickness rather than settlement in inches or feet if a variable is to be evaluated for different lift thicknesses.

128. In evaluating the number of roller passes, economics necessarily enters in; that is, a true optimum based on performance alone can rarely be selected. Rather, one should consider the relative amount of settlement or compaction obtained by additional passes (as indicated by the slope of the settlement-number of passes curve) and, coupled with visual observations of the inspection trench and results of density and percolation tests, make a judgment based on the highest return for the effort put forth and the needs of the embankment, i.e., height, seismic risk, etc. More compaction can almost always be attained by more effort, but the price that must be paid to get it becomes increasingly high.

129. In considering the density data, it is well to keep in mind that a higher density does not necessarily mean a better rock fill. (This in no way precludes the in situ density test as a means of control during construction.) The degree of rock-to-rock contact must also be used to judge a rock fill. This is because high densities of rock fill may mean that the voids between the larger rock are filled (either loosely or tightly) with smaller sized rock and quantities of fines. This can be

detrimental from the standpoint of post-construction settlements and permeability because too many fines can prevent the degree of contact between the larger rock particles needed for stability and can greatly reduce the permeability or drainage characteristics of the fill. Hence, in analyzing density data, the results of each test must be considered in respect to visual observations from the inspection trench as well as settlement data.

130. Final judgments as to the values of the variables evaluated that should be applied to the design and construction of the prototype fill have to be made with consideration for all pertinent measured quantities and observed qualities coupled with a large amount of engineering judgment and experience.

APPENDIX A

SUMMARY OF ROCK TEST
FILL DATA FROM OTHER AGENCIES

1. In order to make this report as comprehensive as possible, several private agencies and other Governmental agencies were asked for any information and data they might have on compacted rock test fills. Those queried were:

a. Private agencies

(1) Bechtel Corporation

(2) EBASCO Services, Inc.

(3) Harza Engineering Company

(4) Mueser, Rutledge, Wentworth, and Johnston

(5) Pacific Gas & Electric Company

(6) Stone & Webster Engineering Corporation

(7) Tippetts-Abbett-McCarthy-Stratton

b. Governmental agencies

(1) California Department of Water Resources

(2) Tennessee Valley Authority (TVA)

(3) U. S. Bureau of Reclamation

2. Replies were received from all of the above except Bechtel Corporation, but only two (EBASCO and TVA) had data that were available at the time. The information from EBASCO on Keban Dam and Hydroelectric Project, Turkey, and from TVA on Tims Ford Dam, Tenn., is briefly summarized in tables A1 and A2, respectively.

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Table A1

Keban Dam and Hydroelectric Project, Turkey*

Rock

Type - Limestone, metamorphic crystalline (faulted and highly fractured with solution channels and clay-filled cavities)

Breakage - Gradation of material finer than 24 in. seemed to depend primarily on the fracture pattern within the rock mass. Changing the blasting patterns and powder factors seemed to affect only the amount of material larger than 24 in.

Gradation before and after rolling - Tests only after rolling. Typical gradations listed below:

Sieve Size	Percent Passing 0.6 meter lift	Percent Passing 1.2 meter lift
24 in.	100	85
12 in.	85	70
5 in.	65	55
2 in.	35	30
1 in.	18	15
0.5 in.	10	9
No. 4	6	5
No. 30	3	2
No. 200	0.7	0.6

Test fill construction

Test fill performed using 2400 kg of material with following gradation:

Sieve Size	Percent Passing
12 in.	75
3 in.	25
1 in.	0

Material was placed in the embankment and rolled with 2, 4, and 6 passes of vibratory roller. After rolling, material was tested for

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* Information on Keban Dam and Hydroelectric Project, Turkey, courtesy EBASCO Services Inc., 2 Rector St., New York, N. Y. 10006. Dam is 299 meters high and was under construction in 1970.

Table A1 Continued

percent passing 1-in. sieve with following results:

No. Passes	Percent Finer Than 1 in.
2	1.5
4	2.2
6	3.6

Compaction equipment

Bros VP-20 Vibra-Packor

Static weight:	20,225 lb
Drum width:	78 in.
Drum diam:	60 in.
Vibration range:	1300 to 1500 vpm
Vibrating force:	40,000 lb

D-8 and D-9 tractors used for spreading rock; D-7 tractor used to tow single drum vibratory roller

Density data

Test method: Water volume. 2-meter-diameter test hole excavated through completed lift, and plastic sheet used to line the hole. Hole volume for 0.6-meter lift was about 1.6 cubic meters, and for 1.2-meter lift was 3.2 cubic meters.

Number of tests: 76 to date. 61 in 0.6-meter lifts and 15 in 1.6-meter lifts.

Results: Average density of 2.21 tons per cubic meter for 0.6-meter lift and 2.08 tons per cubic meter for 1.2-meter lift, with resulting void ratios of 0.22 and 0.28, respectively.

Settlement data

None (cross-arm settlement gages will be installed in the core, and settlement monuments will be installed in the rock fill).

Test trenches

Excavated at random throughout the fill wherever excessive surface powdering indicated that fill contained too many fines. Visual inspections were made, and large-scale samples for gradation were obtained when necessary.

Table A1 Continued

Miscellaneous

Foundation treatment - Foundation rock is limestone-marble underlain by schist with many faults and caverns in the foundation area. Within core area, fault zones and brecciated areas were cleaned out as deeply as practical, depending on width, and backfilled with concrete. In very wide brecciated zones, a concrete pavement was placed at the base of the core across the full width of the core. All caverns were cleaned out and backfilled with concrete. Consolidation grouting was then carried out over the full width of the core to a depth of 10 meters or more, as necessary. In the foundation area within the rock-fill zones, all badly weathered or loose rock was removed and caverns were cleaned out and backfilled with concrete.

Rock-fill compaction - The embankment has an interior rock-fill zone constructed of 0.6-meter lifts and an outer rock-fill zone constructed of 1.2-meter lifts. Both zones were compacted with four passes of the vibratory roller.

Table A2

Tims Ford Dam, Elk River, Winchester, Tenn.*

Rock

Type - Limestone with scattered interbedded shale

Breakage - Maximum rock size limited to 24 in. with only occasional pieces exceeding this size. Several adjustments in the blasting technique were necessary to limit this rock size to 24 in.

Gradation before and after compaction - Tests made before and after rolling; average gradations are listed below:

Screen Size	Before Rolling % Passing	After Rolling, % Passing 3-ft Lift	After Rolling, % Passing 4-ft Lift
24 in.	98	98	96
12 in.	82	94	89
6 in.	65	80	77
3 in.	35	60	55
1-1/2 in.	22	38	36
No. 4	4	8	8

The largest fluctuations occurred in the percent passing the 3-in. screen.

Note: Surface rock, after compaction, to a depth of approximately 15 in. on both the 3- and 4-ft lifts showed a breakdown to a maximum size of about 6 in. In the 3-ft lift, the breakage was more pronounced and transgressed into the lower fill portions:

Screen Size	Average Percent Degradation
12 in.	10
6 in.	13
3 in.	22
1-1/2 in.	15
No. 4	4

* Information on Tims Ford Dam, Winchester, Tenn., courtesy TVA. Tims Ford Dam was started originally as a concrete gravity dam, but changed to earth-rockfill dam in July 1967 because of horizontal clay seams in rock foundation.

Table A2 Continued

No abrasion tests were performed.

Test fill construction

Test fills were made using material excavated from the trench for the power conduit. Test fills were approximately 50 by 60 ft in area, using 2-, 3-, 4-, and 6-ft lifts. The number of passes of the vibrating roller was varied from 1 to 4 for 2-ft lifts, up to 5 for 3- and 4-ft lifts, and up to 8 for 6-ft lifts.

Compaction equipment

Vibro-Plus CT-60

Static weight:	25,000 lb (total weight including tractor = 50,500 lb)
Drum width:	82 in.
Drum diam:	64 in.
Vibration range:	Up to 1250 vpm
Centrifugal force:	30 tons

D-8 tractor used for spreading rock

Density data

Test method: Water volume. See Appendix C

Results: A range of in-place densities from 123.3 to 130.8 pcf was established for 3- and 4-ft lifts with two to four passes. The average in-place density was determined to be 127.3 pcf, and the mean density 127.2 pcf.

Settlement data

A grid system of 20 points marked with spray paint was laid out on each test section and elevations taken before and after rolling. The settlement was then computed based upon lift thickness of each pass of the roller. The results were plotted on a graph showing settlement versus number of passes. Lift thicknesses of 3 and 4 ft were selected for detail testing. Settlement for the 3-ft lift with two passes and the 4-ft lift with three passes was approximately 4 percent.

Test trenches

None

Table A2 Concluded

Rock-fill compaction

The embankment has an upstream rock-fill zone and a downstream rock-fill zone constructed of 3-ft lifts, except that the first lift placed on the foundation and other specified areas were 2-ft lifts. All lifts were compacted by two passes of a 15-ton vibratory roller at 1.5 mph (maximum).

APPENDIX B

USE OF THE LOS ANGELES ABRASION TEST TO PREDICT DEGRADATION

General

1. At the outset of this study, one of the objectives was to compare rock degradation resulting from actual handling and compaction to the degradation obtained in the Los Angeles (LA) abrasion test to determine if a correlation between the two could be established. The field degradation was to be obtained by comparing gradation curves from samples taken before handling the rock and after compaction of the rock. The LA abrasion test results were to be obtained from the field and supplemented by additional tests performed at WES. However, due to the scarcity of before and after gradation data and the variability of the existing abrasion data, the aforementioned objective could not be met.

Before-Handling and After-Compaction Gradation Curves

2. Both before- and after-compaction gradation curves were available on only six of the 14 CE test fills studied. Of these six, one project had no LA abrasion test data to which the curves could be compared, and the gradation curves from another were from rock that was too soft for the LA test to be applicable. The gradations from the remaining four test fills are shown in plates B1 through B5.

Los Angeles Abrasion Tests

3. A summary of the LA abrasion test data is given in table B1.

As is evidenced from the table, a considerable amount of data is available, but there also exists a large variability within the data, caused not only by the fact that there are many different rock types but also by a wide range of allowable gradations on which the test can be run and still meet the ASTM requirements. To begin with, there are two different LA abrasion tests as specified by ASTM: (a) ASTM C131-64T for small-size coarse aggregate (1-1/2-in. to No. 8 sieve) and (b) ASTM C535-65 for large-size coarse aggregate (3-in. to 3/4-in. sieve). In addition, there are four allowable gradings for (a) above and three for (b). Both tests specify that the percent loss be measured using the No. 12 sieve, but ASTM C131-64T specified that it be measured after 500 revolutions of the testing machine, while ASTM C535-65 required 1000 revolutions. As seen in table B1, the LA abrasion tests were run using either method on various gradings within each specified method. This prevented any direct comparison of the results.

Test Results

4. It will be noted that plates B1 and B2 not only contain gradation curves before and after compaction, but also a gradation curve after LA abrasion testing. This is because the LA abrasion tests for this particular test fill were run at WES specifically for this study and complete after-test gradations were determined. It is felt this might be more meaningful than just a percent loss value as measured on the No. 12 sieve, which is the end result specified by the ASTM procedure.

5. Although very limited, the before and after gradation curves shown in plates B1 through B5 give an indication of the breakage due to handling and compaction for a wide range of rock types from a partially weathered shale (plate B1) on one end to a fresh quartzite (plate B3) on

the other. In this respect, it should be noted that the metasandstone represented by the curves in plate B4 is nearer in quality to a quartzitic-type rock rather than the normal sedimentary sandstone (such as was used at Laurel Dam), which normally breaks down much more readily than does a quartzite.

6. The LA abrasion test results for each type rock are also given in plates B1 through B5 and appear to indicate the breakage resistance due to handling and compaction, even though the field compactive effort (defined by a given number of passes over a given loose lift thickness by a specific piece of equipment) varied in all cases. The only rock that the LA abrasion test gave an erroneous indication of the breakage characteristics was the sound limestone from Stockton Dam (see plate B5). A very high percent loss (approximately 42 percent, comparable to the partially weathered shale, plate B2) was obtained from the LA abrasion test; while the gradation curves show the breakage was in fact a minimum (comparable to the quartzitic-type rocks shown in plates B3 and B4). This fact was also confirmed by visual observations in the field.

Conclusions

7. The LA abrasion test will give an indication of the breakage characteristics of a rock. However, depending on how the test is run, this may not be any more of an indication than what could be obtained from a visual examination by a geologist.

8. It is suggested that in the future if a LA abrasion test is run for the purposes of evaluating the breakage characteristics of potential rock fill, the following be given consideration:

- a. The test be run as specified by ASTM C535-65 and on the coarsest grading allowable (grading 1, 3 in. to 1-1/2 in.).

b. A gradation test be performed on all the material after the test.

c. A sufficient number of tests be run so as to make the results more meaningful.

Table B1

Summary of Los Angeles Abrasion Test Results

Project	Rock Type	Test Method		Grading Tested		Results
		ASTM Designation	Revolutions	Maximum	Minimum	(% loss on the No. 12 Sieve)
Laurel	Hard sandstone	C535-65	1000	3 in.	1-1/2 in.	50.9
		C535-65	1000	2 in.	1 in.	58.1
Beltzville	Partially weathered shale	C535-65	1000	3 in.	1-1/2 in.	44.3
	Fresh shale	C535-65	1000	3 in.	1-1/2 in.	30.7
Carters	Quartzite	C535-65	1000	2 in.	1 in.	18.0
	Argillite	C535-65	1000	3 in.	[illegible] in.	23.0
	Phyllite	C535-65	1000	3 in.	1-1/2 in.	35.0
Cougar	Basalt	C131-64T	500	1-1/2 in.	3/8 in.	21.0
Gathright	Sound limestone	*	*	*	*	26.0
Fall Creek	Porphyrite basalt	C535-65	500	3 in.	1-1/2 in.	9.5
	Amygdaloidal basalt	C535-65	500	3 in.	1-1/2 in.	12.5
	Amygdaloidal basalt	C535-65	500	3 in.	1-1/2 in.	11.8
Fukuji	Weathered limestone	C131-64T	500	1-1/2 in.	3/8 in.	33.3
		C131-64T	500	3/4 in.	3/8 in.	32.9
		C131-64T	500	3/8 in.	No. 4	30.4
		C131-64T	500	No. 4	No. 8	29.2
		C535-65	1000	3 in.	1-1/2 in.	36.4
		C535-65	1000	2 in.	1 in.	34.0
		C535-65	1000	1-1/2 in.	3/4 in.	33.5
New Melones	Fresh metasandstone	C131-64T	500	1-1/2 in.	3/8 in.	15.0
	Weathered metasandstone	C131-64T	500	1-1/2 in.	3/8 in.	20.0
Stockton	Sound limestone	C131-64T	500	Crushed		29.5
		C131-64T	500	drill		33.6
		C131-64T	500	cores		38.6
		C131-64T	500	No. 4	3/4 in.	39.7
		C131-64T	500	3/4 in.	1-1/2 in.	42.5
		C131-64T	500	minus 1-1/2 in.		36.0
		C131-64T	500	minus 3/4 in.		35.0
		C131-64T	500	No. 4	3/4 in.	43.5
		C131-64T	500	3/4 in.	1-1/2 in.	44.8
New Hogan	Metavolcanic	C131-64T	500	1-1/2 in.	3/8 in.	13.0
		C131-64T	500	3/4 in.	3/8 in.	12.0
Paint Creek	Hard dolomite	C131-64T	500	*	*	25.0
		C131-64T	500	*	*	27.0
New Hope	Weathered metavolcanic	C535-65	1000	3 in.	1-1/2 in.	26.0
		C535-65	1000	3 in.	1-1/2 in.	33.0
		C535-65	1000	3 in.	1-1/2 in.	31.0

* Information not available

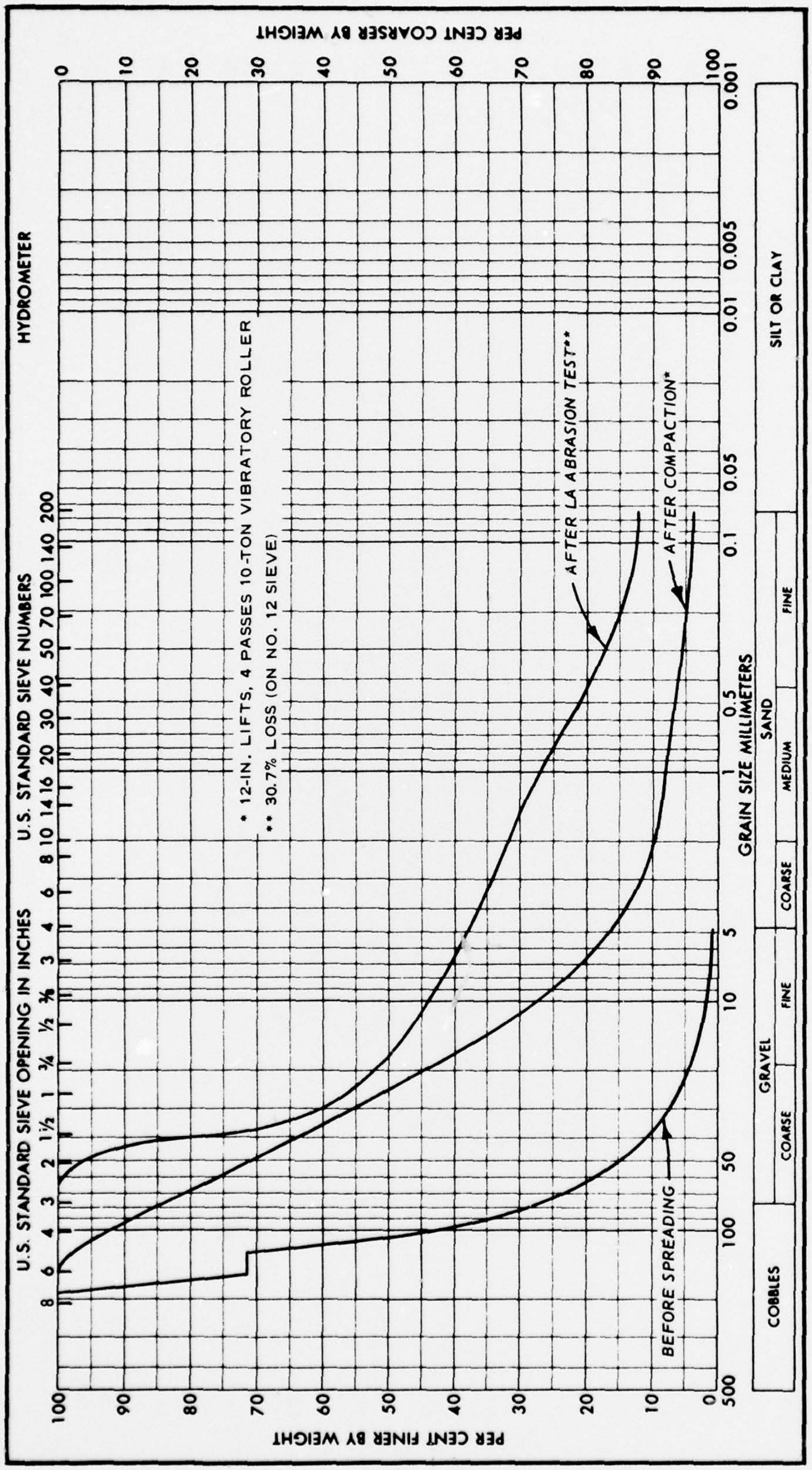
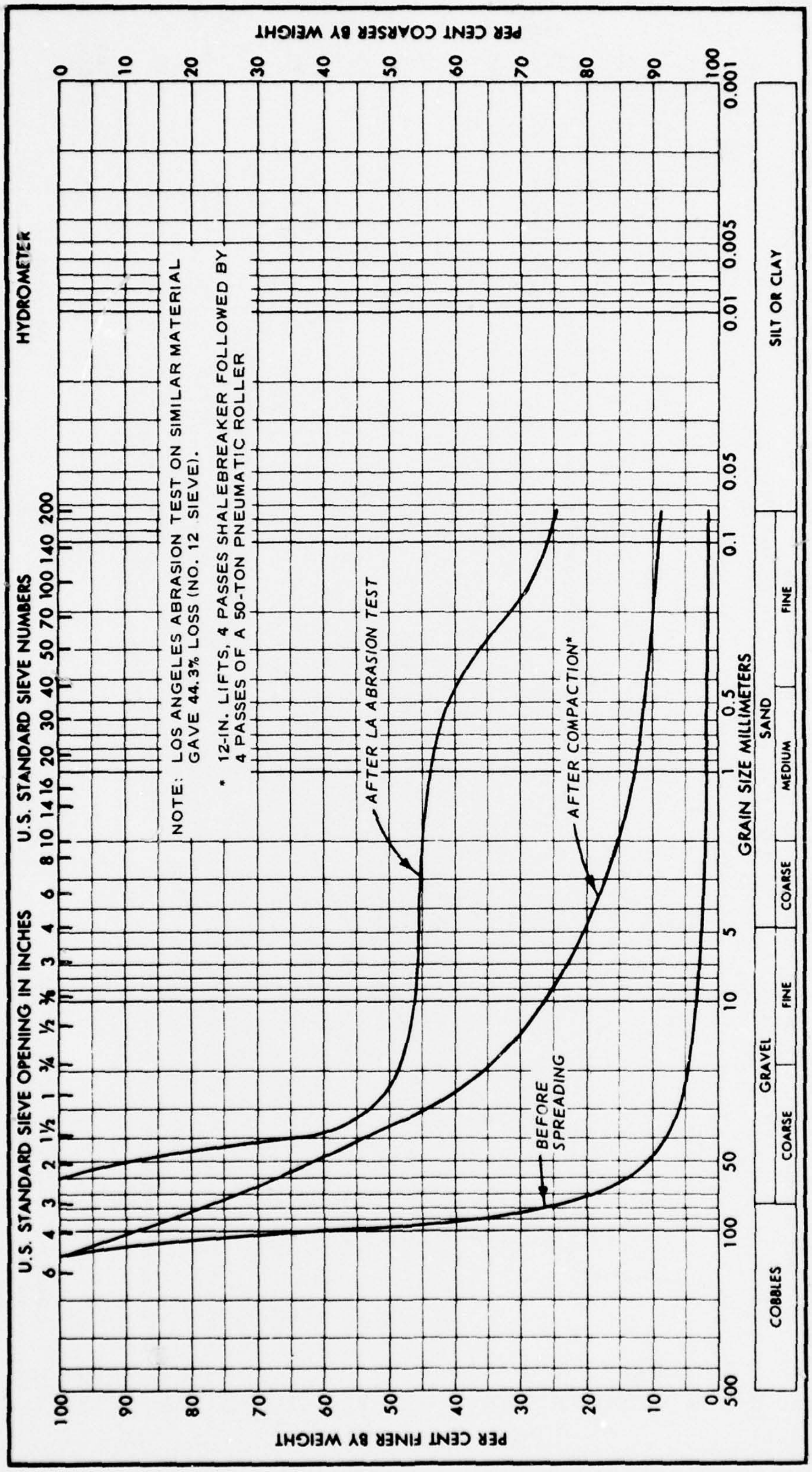

U.S. STANDARD SIEVE OPENING IN INCHES
U.S. STANDARD SIEVE NUMBERS
HYDROMETER
PER CENT COARSER BY WEIGHT
PER CENT FINER BY WEIGHT
GRAIN SIZE MILLIMETERS
* 12-IN. LIFTS, 4 PASSES 10-TON VIBRATORY ROLLER
** 30.7% LOSS (ON NO. 12 SIEVE)
AFTER LA ABRASION TEST**
AFTER COMPACTION*
BEFORE SPREADING
COBBLES
GRAVEL
COARSE
FINE
SAND
COARSE
MEDIUM
FINE
SILT OR CLAY

PLATE B1

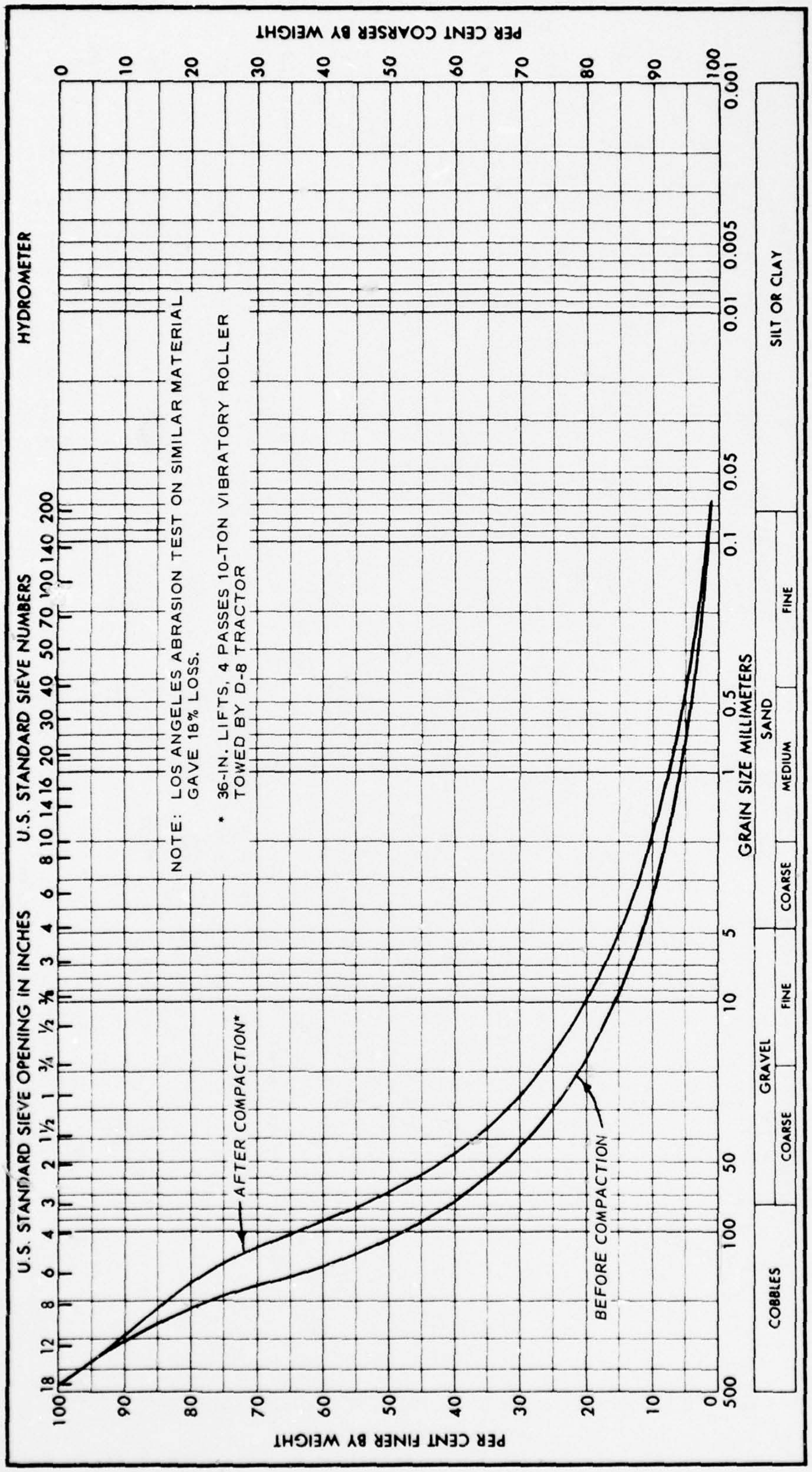
150<



U.S. STANDARD SIEVE OPENING IN INCHES
U.S. STANDARD SIEVE NUMBERS
HYDROMETER
6 4 3 2 1½ 1 ¾ ½ ⅜ 3
4 6 8 10 14 16 20 30 40 50 70 100 140 200
NOTE: LOS ANGELES ABRASION TEST ON SIMILAR MATERIAL GAVE 44.3% LOSS (NO. 12 SIEVE).
* 12-IN. LIFTS, 4 PASSES SHALEBREAKER FOLLOWED BY 4 PASSES OF A 50-TON PNEUMATIC ROLLER
AFTER LA ABRASION TEST
AFTER COMPACTION*
BEFORE SPREADING
PER CENT FINER BY WEIGHT
100 90 80 70 60 50 40 30 20 10 0
PER CENT COARSER BY WEIGHT
0 10 20 30 40 50 60 70 80 90 100
500 100 50 10 5 1 0.5 0.1 0.05 0.01 0.005 0.001
GRAIN SIZE MILLIMETERS
COBBLES
GRAVEL
COARSE
FINE
SAND
COARSE
MEDIUM
FINE
SILT OR CLAY

151<

PLATE B2



U.S. STANDARD SIEVE OPENING IN INCHES
U.S. STANDARD SIEVE NUMBERS
HYDROMETER
18 12 8 6 4 3 2 1½ 1 ¾ ½ ⅜ 3 4 6 8 10 14 16 20 30 40 50 70 100 140 200
PER CENT FINER BY WEIGHT
100 90 80 70 60 50 40 30 20 10 0
PER CENT COARSER BY WEIGHT
0 10 20 30 40 50 60 70 80 90 100
NOTE: LOS ANGELES ABRASION TEST ON SIMILAR MATERIAL GAVE 18% LOSS.
* 36-IN. LIFTS, 4 PASSES 10-TON VIBRATORY ROLLER TOWED BY D-8 TRACTOR
AFTER COMPACTION*
BEFORE COMPACTION
500 100 50 10 5 1 0.5 0.1 0.05 0.01 0.005 0.001
GRAIN SIZE MILLIMETERS
COBBLES
GRAVEL
COARSE
FINE
SAND
COARSE
MEDIUM
FINE
SILT OR CLAY

152<

PLATE B3

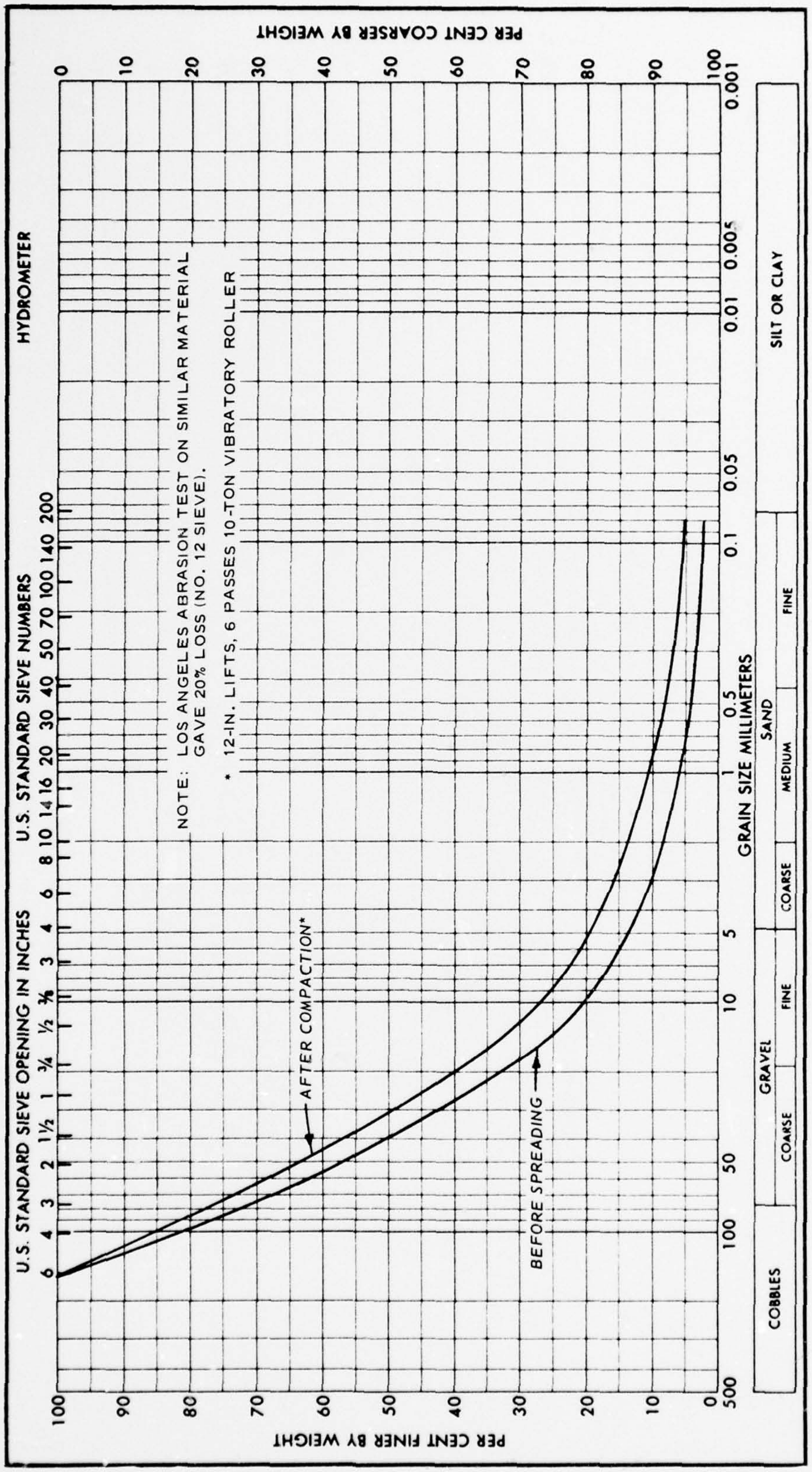
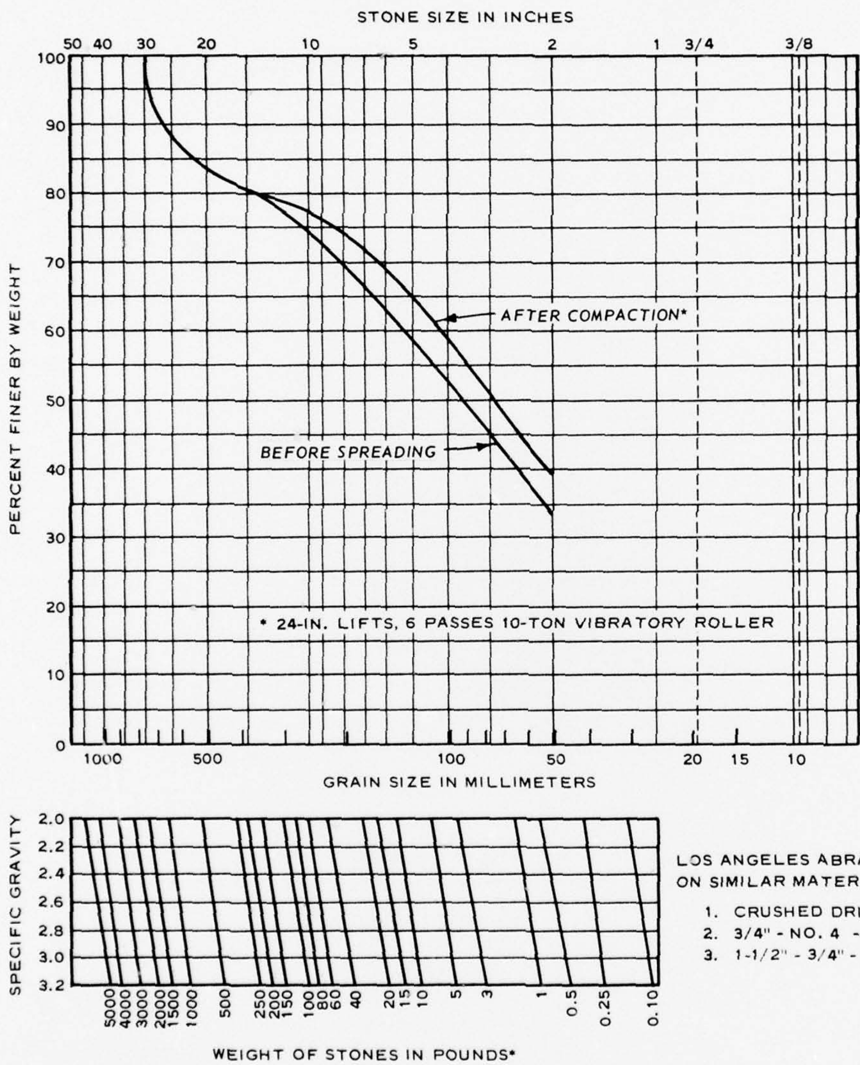

U.S. STANDARD SIEVE OPENING IN INCHES
U.S. STANDARD SIEVE NUMBERS
HYDROMETER
6 4 3 2 1½ 1 ¾ ½ ⅜ 3 4 6 8 10 14 16 20 30 40 50 70 100 140 200
PER CENT COARSER BY WEIGHT
0 10 20 30 40 50 60 70 80 90 100
PER CENT FINER BY WEIGHT
100 90 80 70 60 50 40 30 20 10 0
500 100 50 10 5 1 0.5 0.1 0.05 0.01 0.005 0.001
GRAIN SIZE MILLIMETERS
NOTE: LOS ANGELES ABRASION TEST ON SIMILAR MATERIAL GAVE 20% LOSS (NO. 12 SIEVE).
* 12-IN. LIFTS, 6 PASSES 10-TON VIBRATORY ROLLER
AFTER COMPACTION*
BEFORE SPREADING
COBBLES
GRAVEL
COARSE
FINE
SAND
COARSE
MEDIUM
FINE
SILT OR CLAY

PLATE B4



LOS ANGELES ABRASION TEST RESULTS ON SIMILAR MATERIAL:

1. CRUSHED DRILL CORES - 34.3%
2. 3/4" - NO. 4 - 41.6%
3. 1-1/2" - 3/4" - 43.7%

* ASSUMING STONE SHAPE MIDWAY BETWEEN A SPHERE AND CUBE.

Gradation curves for riprap filter and bedding, Stockton test fill

APPENDIX C

IN SITU DENSITY TEST IN COMPACTED ROCK FILL

Introduction

1. This test may be performed using either water (confined in plastic sheeting) or sand as the displacement medium and a surface template of either a steel ring or a frame of wooden two-by-fours. However, it is recommended that water be used as the displacement medium and a 6-ft-diam by 8-in. high ring be used as the surface template. Experience has shown that tests employing these items give the most consistent results with a minimum of effort. The test procedure described in the following paragraphs is based on the use of water and a 6-ft-diam steel ring.

Test Procedure

Surface calibration

2. The following steps outline a procedure of surface calibration to account for unevenness of fill surface:

- a. Select an area for the test, avoiding obviously uneven surfaces and locations where removal of large rock would undermine the ring.
- b. Place the 6-ft-diam by 8-in. -high steel ring on the fill surface at the desired test location.
- c. Pick out all obviously loose and/or sharp rock fragments protruding from the surface within the ring. Fill any large open spaces where the ring does not contact the fill surface with stiff mortar. Lay three or four sandbags against the outside and slightly over the top of the ring to prevent any movement during the test.
- d. Lay a double thickness of polyethylene sheeting loosely within the ring and overlapping the ring. New polyethylene

sheeting, 4 to 6 mil in thickness should be used.

e. Record an initial reading of the water meter.

f. Run metered water carefully onto the sheeting inside the ring, making sure there is sufficient slack in the sheeting until the water level is 3 to 4 in. below the top flange of the density ring.

g. Record the water meter reading. This reading minus the initial reading will give the volume of calibration water (V_c).

h. Mark the water level distinctly on the inside of the ring at two diametrically opposite points. In windy weather, windbreaks may be necessary to obtain accurate measurements.

i. Remove the water from the ring, being careful not to disturb the ring or tear the sheeting.

j. Remove each layer of sheeting separately, and carefully check each layer for leaks. Discard the sheeting.

Excavation and weight determination

3. The following steps outline a procedure for excavating and weighing material from the test hole:

a. Taking great care not to disturb or undermine the ring, excavate the rock from within the ring to the desired depth (usually one lift thickness). At the outset of the excavation, the walls of the hole should be made as nearly vertical as possible; then, as the desired depth is approached, curve the walls in toward the bottom so that the shape of the final hole is similar to that shown in fig. C1. Remove from the bottom all loose material that can be picked up with a round-point shovel.

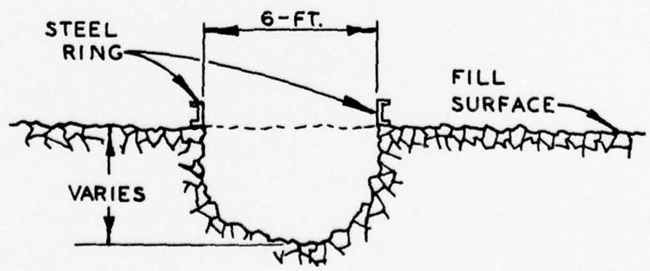


Fig. C1. Excavated density hole

b. During excavation care must be taken not to lose any of the excavated material. This is best accomplished by placing the material in containers such as steel drums that can be weighed to determine the weight of the excavated material (W).

Volume determination

4. The following steps outline a procedure for measuring the volume of the excavated hole:

a. The volume of the hole is determined by lining it with polyethylene sheeting and measuring the volume of water required to fill it. Prior to lining the hole, all deep recesses (into which the sheeting would not depress, even under the pressure of the water) in the walls and bottom of the hole left by rock removal should be hand packed with stiff cement mortar. Also, mortar should be hand-packed around any sharp rock points that may puncture the sheeting. The volume of mortar needed for the above operations must be measured and recorded (V_m). If cavities not due to rock removal are noted in the floor or walls of the hole, they should be filled with unmeasured mortar to the general outline of the hole.

b. The hole is now ready to be lined with a double thickness of new 6-mil thick polyethylene sheeting. Prior to placing the sheeting in the hole, it should be carefully inspected for punctures. Without necessarily waiting for the mortar to set, line the hole with the sheeting, which should be large enough to extend 2 to 3 ft outside the ring after having been carefully shaped within the hole.

c. Slowly fill the hole with metered water while at the same time shifting the sheeting around the sides of the hole to provide slack for the pressure of the water to force the sheeting into recesses and around projecting rocks. The sheeting should never be stretched too taut and care must be taken in the above shaping process in order to obtain a realistic volume determination.

d. Continue filling with water until the previously marked calibration lines are reached. If the water does not reach both calibration marks simultaneously (this would be because of disturbance of the ring), bring the water level to a level midway between the two marks. Record the volume of water used to fill to this level (V_w).

e. Remove the water from the hole. Again carefully check the sheeting for punctures; if no punctures are present, remove the sheeting and steel ring, backfill the hole with new rock fill from placement operations, and compact with the roller being used on the job. If a puncture in the membrane is detected, check the hole to see if substantial leakage occurred; if so, the hole must be relined and the volume measurement repeated.

Density determination

5. The density is computed from the following equation:

$$\gamma = \frac{W}{V_w + V_m - V_c} \tag{C1}$$

where

γ = moist unit weight of in-place rock fill, pcf

W = weight of material removed, lb

V_w = volume of water required to fill the hole, cu ft

V_m = volume of mortar required to fill in recesses and placed around sharp projections, cu ft

V_c = volume of water used in surface calibration, cu ft

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